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STANDARDS DEVELOPMENT BRANCH OMOE
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WASTEWATER TREATMENT PLANT UPGRADING

MARCH 10, 1982

PROCEEDINGS

Organized by:

POLLUTION CONTROL ASSOCIATION OF ONTARIO

ONTARIO MINISTRY OF THE ENVIRONMENT

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POLLUTION CONTROL ASSOCIATION OF ONTARIO
and the
ONTARIO MINISTRY OF THE ENVIRONMENT

PROCEEDINGS
# of the
SEMINAR on

WASTEWATER TREATMENT PLANT UPGRADING

March 10, 1982

held at the
Ontario Ministry of Health Laboratories
Resources Road, Toronto

Seminar Co-Chairmen: E.E. Shannon,
Canviro Consultants Ltd.

C.B. Gray,
The Gray Engineering Group

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COMPUTERIZED FLOW MONITORING
IN
STRATFORD, ONTARIO

BY

STEPHEN A. PHILLIPS
AMERICAN DIGITAL SYSTEMS INC.

Over the years, measurement of flow in sewage systems has proven to be extremely difficult. Aside from the obvious drawbacks resulting from the presence of a hostile environment, numerous problems are commonly encountered when trying to accurately measure flows in sewers.

Many of the problems faced are inherent in open channel flow characteristics. Others are not associated with hydraulic considerations but develop from the need to acquire, store, and analyze masses of data produced from flow monitoring programs designed to provide an analysis of a given sewage system or subsystem.

The City of Stratford, Ontario completed a hydraulic analysis of approximately 354,800 linear feet of sewers in June of 1981 using flow monitoring techniques and equipment which minimize the general problems of open channel flow characteristics and data management. This discussion describes the overall approach to the project, the specific equipment used, and the results obtained.

An earlier storm and wastewater control study performed in 1972 cited the most serious problems in the Stratford Collection System as being:

- Surcharging in 24" and 48" sewers along the shore of Lake Victoria and approaching the sewage treatment plant;
- Bypassing into the Avon River in excess of 17 MGD of combined storm and sanitary sewage during intense rainfall;
- Resident complaints and samplings of storm water outfalls along Lake Victoria and the Avon River.

The system is divided by 3 major outfalls: a 24" outfall from the north, a 30" outfall from the east, and parallel 48" outfall from the northeast.

The objective of the recent computerized flow monitoring study in Stratford was to isolate those areas in the system with the worst inflow and infiltration problems.

The overall approach to the project developed from the answers to several questions. The first of these is illustrated in Figure 1 - "What to Measure?". As can be seen, depth, no matter how measured, is the most critical parameter in determining open channel flow quantities. Parameters such as pipe roughness, slope, or weir coefficients will yield errors in total flow (Q) of between 3% and 10% if these parameters are measured with a 10% error. However, a 10% error in depth of flow measurement can result in a 25% - 70% error in Q, depending upon how the depth of flow figure is being obtained. Thus, an accurate, consistent and reliable depth of flow is required in order to provide accurate, consistent and reliable quantities of flow.

The next significant question considered in the project was "How to Measure?" (see Figure 2). All flow monitoring devices have 3 basic components: the sensor, the timing mechanism, and the data recording device. An examination of the basic components of typical electromechanical devices is provided in Figure 2. It is apparent that flow monitoring equipment utilizing the "mechanical approach" is subject to significant error in depth of flow measurement. As discussed above, depth of flow measurement error results in inaccurate flows, providing meaningless data. The City of Stratford, however, chose to use the electronic (computerized) approach described in the right column of Figure 2. As can be seen by comparison, the electronic approach overcomes many of the equipment limitations described in the "mechanical approach" column. Rather than a float-type sensor with a servo-mechanism (or a capacitance probe and a servo-mechanism, a compressed gas back pressure system, etc.) the approach taken by Stratford used solid-state pressure transducers to sense hydrostatic head (which is easily converted to depth of flow). This transducer is accurate to ± 1/8 inch depth. The timing mechanism, critical to correlation of data, used in the Stratford project, was a quartz clock with ± 5 seconds/month accuracy. In a large project, limitations of mechanical clocks used in most types of flow monitors can reduce the data to a meaningless labyrinth. The quartz timer permits the development of a "hydraulic snapshot" of the entire system under study within ± 5 seconds/month each time a depth of flow reading is taken. This is an invaluable tool in conducting the data analysis. Other advantages of the quartz timer are that it is less prone to failure than a mechanical clock, and can be accurately synchronized to all timers in the study by use of a master quartz "synchronizer".

The third component illustrated in Figure 2 is one that has both an obvious and a subtle impact on the accuracy, reliability, and consistency of the data.

The obvious impact is that the mechanical features of a strip chart inherently produce error. Smeared ink, dry pens, wet charts, broken pens, etc., all obviously produce errors. The subtle impact of strip charts on the data is the often negative results due to the human interface required in collecting, reading, interpreting and recording extensive data from strip charts. These latter steps often become overwhelming in a study of any magnitude and are sources of considerably varying interpretations. The approach used on the Stratford project, however, eliminates the negative impacts of strip charts mentioned above. Instead of strip charts, each flow monitor used a computer memory to record the data. The data was stored in memory in digital form and retrieved when needed. Being stored in this manner totally eliminates the problems of strip charts mentioned above. The data points are not subject to interpretations or extrapolation. Data storage capacity is such that physically handling the data becomes an easily managed task. This also facilitates use of computers for performing data analysis.

Figure 3 illustrates the final consideration in developing the approach used in the project. Placement of the depth of flow sensor is critical to the accuracy of the depth of flow measurements. If the sensor is placed in the invert of the manhole, as has been common practice, accurate depth measurement is nearly impossible. This results from the turbulence created by the normal 1/10 foot elevation drop across the manhole. Also, manhole inverts seldomly have uniform cross-sectional areas. This prevents the use of most open-channel flow equations which are heavily dependent upon geometric variables. It is possible to overcome these two problems by the use of a primary device such as a weir or flume. In most applications, these devices are not practical due to leakage, heavy siltation and obstruction of weir edges by debris. Also weirs can cause undesirable flow restrictions. The approach used in the Stratford project avoided these complications. The sensors used were placed up in the incoming pipe as illustrated in Figure 3. The distance at which the sensors were placed was 2 to 3 pipe diameters from the inlet point. The sensors are mounted on expandable rings of nominal pipe diameter which can be securely locked into place when the sensor location is selected. This approach minimizes the effect of turbulence, makes measurement in uniform cross section possible, and avoids flow restrictions. The width of the ring and shape of the sensor are designed to impact negligible flow disturbance.

This method of sensor placement lends itself to the use of "calibrated" Manning's equation for determining quantity of flow based on sequential depth of flow measurements. This is accomplished in the following manner:

At each monitoring point, a series of instantaneous velocity and depth of flow measurements are made during the project. Measurements are taken throughout the diurnal pattern so as to include the entire spectrum of flow characteristics present at a given site. Using the measured pipe diameter and series of velocity and depth of flow measurement, a depth-to-discharge curve is developed. This is done by solving Manning's equation for the quantity $\frac{\sqrt{s}}{n}$ for each velocity and depth of flow measurement taken. The velocity used in the equation is 0.9 x Peak Velocity measured in the site.

$$V = 1.486 \cdot \frac{\sqrt{s}}{n} \cdot R^{h2/3}$$

where

V = Velocity
s = Slope of pipe at point where measurement is taken
N = Friction coefficient of pipe section where measurement is taken
R^h= Hydraulic radius, a function of pipe diameter and depth of flow

The series of values obtained are used to provide variables for employing the Continuity Equation,

$$Q = AV$$

where Q = Quantity of flow
A = Cross-sectional area of flow (function of measured pipe diameter and measured depth of flow)
V = Velocity of flow (extrapolated from series of $\frac{\sqrt{S}}{n}$ calculations)

The actual equipment used in the Stratford project consists of three major components: the sensor and ring, the memory and timer, and the interrogator. As mentioned above, the sensor is a pressure transducer. This transducer senses the changes in hydrostatic head by the changing pressure on a silicon wafer which transmits an impulse to the memory. The inlet port has a fine screen on the opening in order to prevent fouling by silt and debris. In addition, the hydraulic profile of the sensor is such that debris accumulation on the sensor is minimized.

The quartz timer in each monitor is synchronized to every other monitor in the project by a master timer. In the Stratford project, these timers were set so that depth of flow measurements were taken and recorded every 15 minutes (within ± 5 seconds/month). The memory which stores all the depth data has the capability of storing up to 21 days worth of depth of flow measurements based on 15-minute measurement intervals. Nevertheless, data was retrieved from the monitors a minimum of twice a week in order to minimize lost data due to equipment malfunctions. These twice weekly visits also are used to perform quality control checks, note general observations and to obtain instantaneous velocity and depth of flow measurements described earlier.

The data stored in the memory is retrieved by an interrogator. This device scans the memory in the monitor and retrieves all data points collected since the previous retrieval. The data points are printed on a paper tape which ultimately serves as the source of key punch input to a central computer specially programmed for hydraulic analysis. The paper tape also enables the field technician to observe trends in the data since the last retrieval.

The advantages of computerized flow monitoring become obvious when one considers that in the Stratford project approximately 50,400 data points were collected (25 monitors x 21 days x $\frac{\text{60 minute/hour}}{\text{15 minute/sample}}$ x 24 hours/day).

These data points were reduced to a more manageable form by selecting 6 "data days" for analysis. These 6 data days consisted of the day of a rain event, a subsequent high groundwater (rain-induced infiltration) day, a recovery day (collectively referred to as wet-weather days), and three representative dry days. Nevertheless, a total of 14,400 data points remained for analysis. Dividing the analyzed data into wet and dry data days provides the basis for determination of the effect of a rain event and increased groundwater on the collection system.

A sample of the analyzed data points is presented in Figure 4. A table of this type was prepared for each monitoring point in the Stratford project for each of the 6-data days. As can be seen, this table identifies the specific monitoring site, the pipe diameter, energy gradient, etc. For each of the 6-data days, the quarter-hourly depth of flow and hourly flow are tabulated. In addition incremental and cumulative rainfall amounts are shown. These rainfall amounts were obtained from tipping bucket rain gauges using the same memory and timer as the flow monitors.

Figure 5 is an illustration of a plotted hydrograph using the data points contained in Figure 4. This hydrograph provides quick means of evaluating the impact of rain events in a particular site.

All of these data points are entered into a computer program which provides a summary of key variables as well as performing flow isolation analysis on both inflow and infiltration. The summary is illustrated in Figure 6. As can be seen, the summary lists each monitoring point and provides a direct comparison of dry and wet weather flows. This comparison provides insight into the potential capacity problems that may exist in the subsystem behind each monitor.

The inflow and infiltration flow isolation analysis shown in Figure 7 and 8 respectively, provide an invaluable management tool. In effect, these analyses provide a prioritization of problem areas. The computer program sorts the data and ranks each subsystem in accordance with the reaction to the rain event. In other words, the subsystems exhibiting the greatest flow increases after the rain event are easily identified. This enables the city maintenance department to prioritize its rehabilitation activities. As can be seen from Figures 7 and 8, approximately 81% of the inflow was found to originate in 42% of the system monitored. Likewise, 84% of the infiltration was distributed in 58% of the system. It becomes readily apparent as to where rehabilitation efforts should be concentrated. Figure 9 gives a graphic illustation of these percentages. The shaded areas identify the origin of 81% of the total inflow. Of course, the inflow analysis identified all the inflow but as a practical matter, the 42% of the system containing 81% of the total inflow should be the areas targeted for rainfall simulation and rehabilitation. The cross-hatched areas represents 58% of the system which contains nearly 84% of the infiltration sources. This area is the area which should be targeted for flow isolation and TV and cleaning. These activities will enable the City of Stratford to locate major problem areas within the smallest possible number of reaches.

Thus, one of the major benefits of the study conducted in Stratford is the "finding of the leaks" in lengths of subsystem which are practical to attack for rehabilitation. No longer will Stratford be looking for "needles in the haystacks". Of course, actual quantification of flows is a major benefit as well. This accurate quantification enables the City to plan for sewage treatment plant expansion, relief line sizing, etc.

Figures 10 and 11 illustrate significant relationships between monitor spacing and distribution of inflow/infiltation problems. The families of curves shown are based on 65 projects conducted throughout the United States and England using the same equipment and techniques employed in Stratford.

As can be seen from Figure 10, monitor spacing directly affects the location of inflow sources in the system. As an example, if monitors are located at 15,000-feet intervals, analysis will show that 80% of the inflow sources are located in 50% of the system. However, monitor spacing of 5,000 feet will yield an analysis which locates 80% of the inflow sources in 20% of the system. The implications are that the monitor spacing has a definite impact on the total cost of the overall rehabilitation effort. Having to pinpoint and repair inflow sources in 50% of the system will obviously require greater expenditures than doing so in less than 20% of the system. It will be noted that at the monitor spacing used in Stratford, approximately 14,200 feet, the curves predict that 80% of the total inflow would be located in 50% of the system. Actual results were somewhat better than anticipated (81% in only 42% of the system).

A similar set of curves shows the same type of relationship between monitor spacing and location of infiltration sources. As seen in Figure 11, monitor spacing of 15,000 feet would locate 80% of infiltration in approximately 50% of the system. Monitor spacing of 4,000-feet intervals, however, will locate 80% of the sources in only 20% of the system. Because the location and repair of specific infiltration sources is exteremely expensive, the decision on monitor spacing for determination of infiltration sources is critical. It is interesting to note that the distribution of infiltration sources in the Stratford project falls closely within the curves in Figure 11 when extrapolated to the 84% of infiltration sources and 14,200-feet intervals used.

These curves provide a valuable input into the development of the most cost-effective approach to the needs of a particular community. A cost-effective approach is normally one which locates approximately 80% of the inflow/infiltration sources in the smallest area possible.

As an example, the community planning a complete rehabilitation program can first determine unit costs for flow monitoring, flow isolation, rainfall simulation, cleaning, TV inspection, etc. Using the distribution curves in Figures 10 and 11, a determination can be made on the cost of flow monitoring at given intervals and of the resultant "follow-on activity" costs for the "80% problem" areas targeted by the flow monitoring analysis. Several iterations of cost determinations of this type will yield the most cost-effective approach. Experience in the United States has shown that generally speaking, the most cost-effective monitoring spacing has been 4,000-feet intervals. Very large systems do not lend themselves to this dense spacing however and are most cost effectively approached by a two-step process. The first step is a monitor spacing of 20,000-feet intervals. This spacing will locate 80% of the infiltration sources in 60% of the system. This 60% can then be cost effectively monitored at 4,000-feet intervals. This spacing will locate 80% of inflow in less than 20% of the system. With these types of elimination, typical SSES costs (all program activities less actual rehabilitation) in the United States have been cut in half.

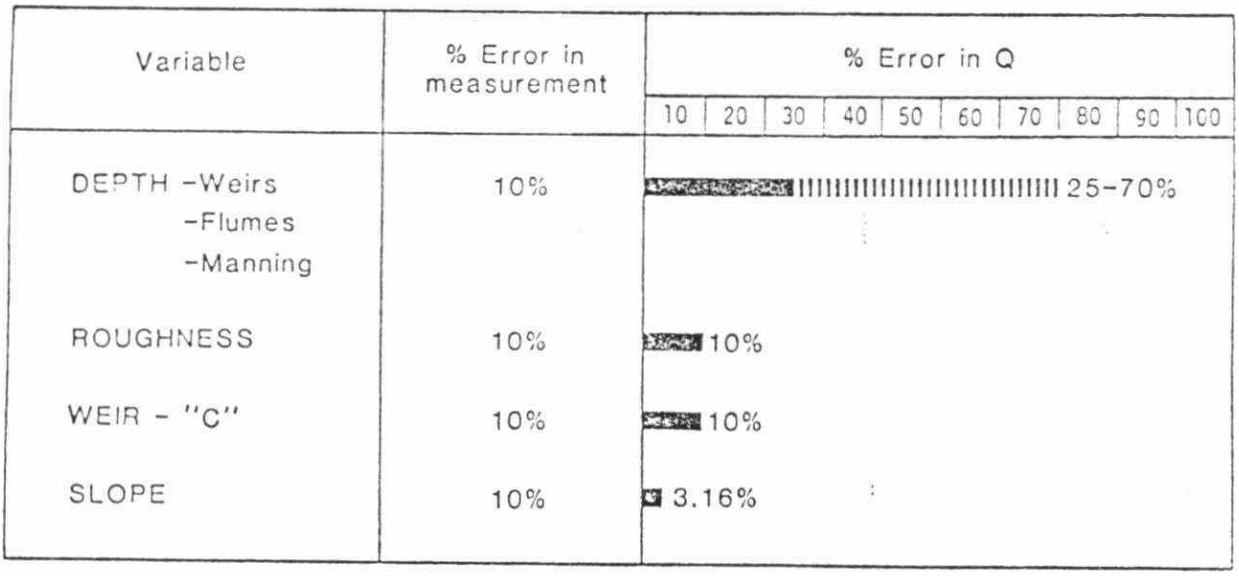
Figure 12 is an example of the cost savings possible. The column on the left represents budgeted costs for a program conducted on the West Coast of the United States. This budget was developed without consideration of the monitor spacing as discussed above. The center column reflects the actual project costs after deciding to use the type of approach discussed above at a monitor spacing of 4,000 feet. The column on the right contains the cost savings realized in this approach. A net savings of $183,000 in a program budgeted for $371,000 is dramatic testimony to the success of the approach used.

Although there may be other suitable approaches to flow monitoring projects, the emergence of solid-state equipment and computerized data analysis has dramatically improved project results in numerous projects in the United States and several in Canada. In conjunction with systematic and consistent monitor site spacing, the approach to flow monitoring described above has demonstrated the ability to not only obtain accurate, reliable and consistent results, but to do so with cost effectiveness especially when the entire program is considered.

FIGURE 1

WHAT TO MEASURE: DEPTH

IN ANY FLOW EQUATION DEPTH IS THE MOST CRITICAL VARIABLE.



Variable	% Error in measurement	% Error in Q
DEPTH -Weirs -Flumes -Manning	10%	25-70%
ROUGHNESS	10%	10%
WEIR - "C"	10%	10%
SLOPE	10%	3.16%

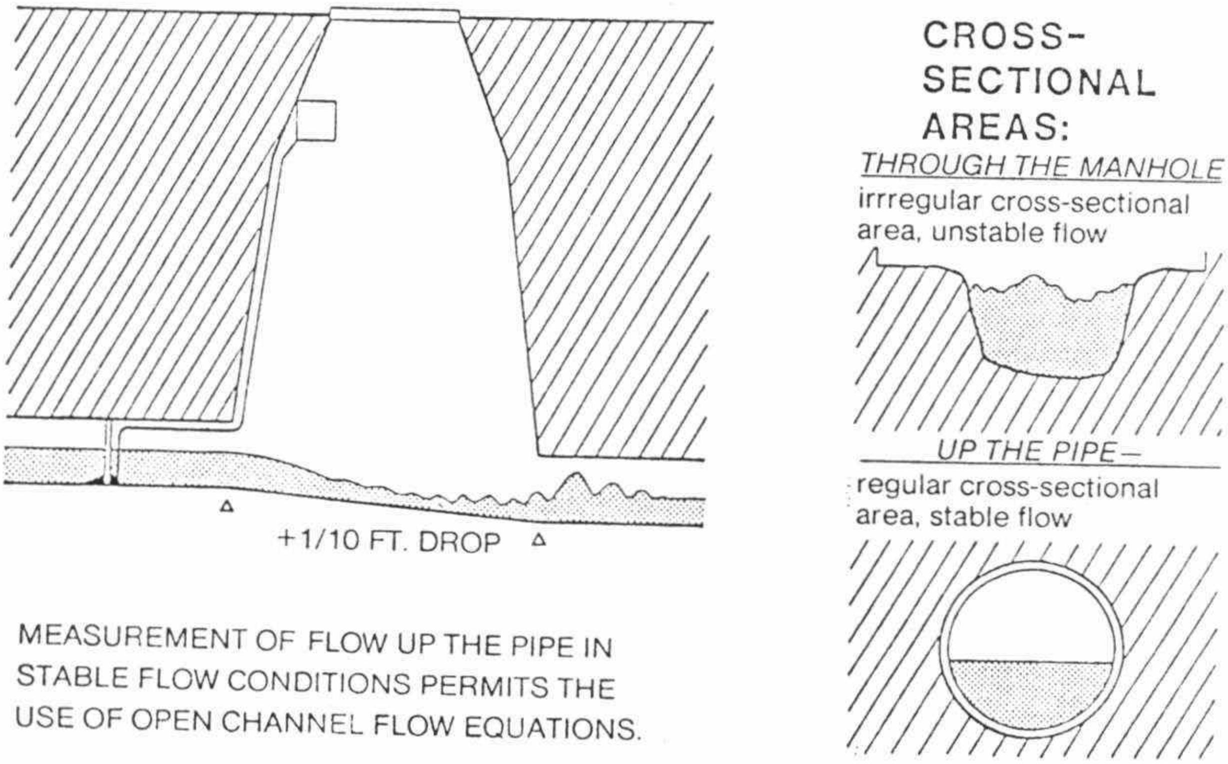
A SLIGHT ERROR IN DEPTH MEASUREMENT (>¼")
CAUSES SUBSTANTIAL ERROR IN "Q".

FIGURE 2

HOW TO MEASURE: EQUIPMENT SPECIFICATIONS

	Mechanical Approach	Electronic Approach
Data Recording Device:	Strip Charts (±1 inch depth)	Computer Memory Bank (±1/8 inch depth)
Timing Mechanism:	Wind-up Key (±30 minutes/day)	Quartz Clock (±5 seconds/month)
Sensor:	Float-type with servo mechanism (±1/2-2 inches depth	Pressure Transducer (±1/8 inch depth)
	RESULTANT DEPTH ERROR ± 3/4 - 2 inches	NEGLIGIBLE DEPTH ERROR ± 1/4 inch

FIGURE 3



WHERE TO MEASURE:

+1/10 FT. DROP
CROSS-SECTIONAL AREAS:
THROUGH THE MANHOLE
irrregular cross-sectional area, unstable flow
UP THE PIPE—
regular cross-sectional area, stable flow

MEASUREMENT OF FLOW UP THE PIPE IN STABLE FLOW CONDITIONS PERMITS THE USE OF OPEN CHANNEL FLOW EQUATIONS.

FIGURE 4

INFILTRATION/INFLOW ANALYSIS OF STRATFORD, ONTARIO

PREPARED FOR THE CITY OF STRATFORD AND THE MINISTRY OF THE ENVIRONMENT
GROSS FLOW MONITORING (PATENT # 4070563)

SITE:H2 21; EQUATION:CAL-MANNING W/VARYING EG; SHAPE:24.000D; ENERGY GRAD:2.750; RAIN ZONES:1,1

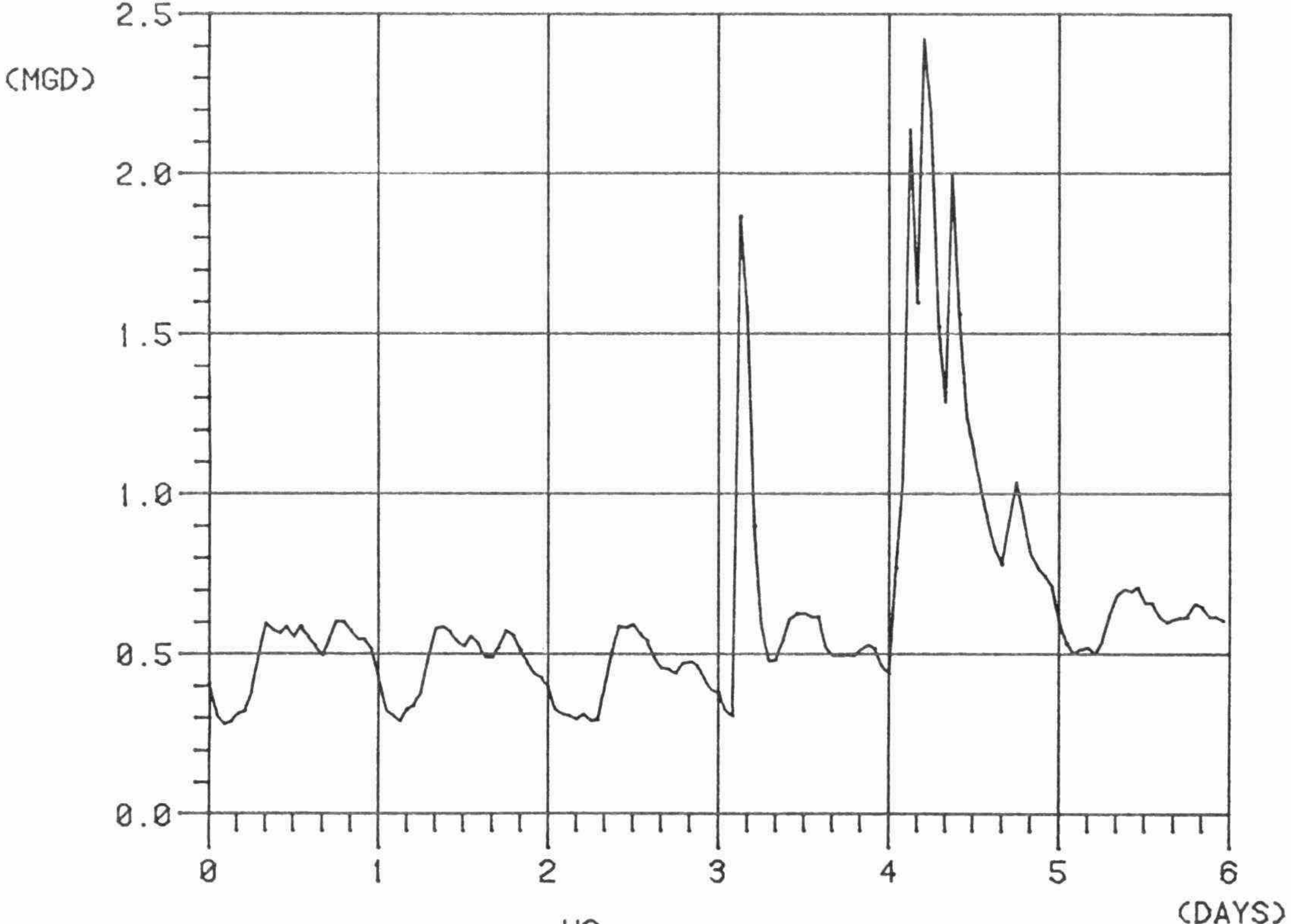
	THURSDAY(1) 6/18/81:DRY			FRIDAY(2) 6/19/81:DRY			SATURDAY(3) 6/20/81:DRY				SUNDAY(4) 6/21/81:WET			MONDAY(5) 6/22/81:WET			TUESDAY(6) 6/23/81:REC		
HR	RAIN (IN)	FLOW (IN)	FLOW (MGD)	RAIN (IN)	FLOW (IN)	FLOW (MGD)	RAIN (IN)	FLOW (IN)	FLOW (MGD)	DRY DAY AVG (MGD)	Z:1 RAIN (IN)	FLOW (IN)	FLOW (MGD)	Z:1 RAIN (IN)	FLOW (IN)	FLOW (MGD)	RAIN (IN)	FLOW (IN)	FLOW (MGD)
0:		5.500	0.404		5.500	0.425		5.375	0.394	0.394		5.000	0.378	1.04	5.375	0.439		6.500	0.604
		5.125			5.500			5.125				5.125			5.375			6.375	
		5.000			5.250			4.875				5.000			5.250			6.125	
		4.875			4.750			4.875				4.750			5.375			6.000	
1:		4.750	0.305		4.750	0.322		4.750	0.327	0.327		4.750	0.322		5.875	0.774		6.125	0.534
		4.375			4.625			4.750				4.375			7.250			6.000	
		4.500			4.500			4.375				4.750			7.625			5.625	
		4.250			4.500			4.625				4.500			7.500			5.750	
2:		4.375	0.279		4.625	0.305		4.500	0.309	0.309		4.500	0.305		7.125	1.043		5.875	0.500
		4.250			4.375			4.625			0.69	4.500			7.125			5.500	
		4.250			4.500			4.500				4.250			8.000			5.750	
		4.250			4.375			4.375				4.625			10.375			5.625	
3:		4.500	0.288		4.125	0.288		4.625	0.305	0.305		5.875	1.865		11.750	2.141		5.750	0.516
		4.250			4.250			4.375				11.625			12.875			5.875	
		4.250			4.375			4.375				13.250			12.250			5.750	
		4.375			4.625			4.500				12.625			11.125			5.750	
4:		4.375	0.313		4.500	0.327		4.375	0.296	0.296		11.125	1.571		10.375	1.596		5.875	0.522
		4.625			4.500			4.375				10.625			9.750			5.750	
		4.625			4.875			4.375				9.750			9.750			5.750	
		4.500			4.625			4.500				9.125			11.125			5.875	
5:		4.500	0.322		4.875	0.340		4.500	0.313	0.313		8.250	0.896		12.125	2.424		5.625	0.499
		4.625			4.625			4.500				7.875			12.875			5.625	
		4.625			4.750			4.500				7.375			13.000			5.750	
		4.625			4.625			4.625				7.000			13.375			5.750	
6:		4.875	0.379		4.750	0.384		4.375	0.292	0.292		6.625	0.598		13.250	2.185		5.750	0.539
		4.625			5.000			4.500				6.250			12.500			5.875	
		5.000			5.000			4.375				6.125			11.500			5.875	
		5.375			5.250			4.250				5.875			11.250			6.125	
7:		5.500	0.488		5.375	0.483		4.250	0.296	0.296		5.625	0.477		10.500	1.521		6.250	0.622
		5.625			5.500			4.375				5.625			10.000			6.375	
		5.625			5.750			4.375				5.500			9.875			6.250	
		5.750			5.750			4.625				5.500			9.625			6.500	
8:		6.000	0.598		6.125	0.580		4.750	0.389	0.389		5.500	0.483		9.375	1.284		6.625	0.683
		6.250			6.125			4.875				5.625		0.17	8.750			6.625	
		6.250			6.125			5.125				5.625			8.875			6.750	
		6.375			6.125			5.375				5.625			9.625			6.625	
9:		6.125	0.574		6.375	0.586		5.500	0.505	0.505		5.875	0.545		12.000	1.995		6.750	0.702
		6.000			6.125			5.625				5.875			11.875			6.625	
		6.250			6.125			5.875				5.875			11.250			6.875	
		6.000			6.000			5.875				6.125			11.125			6.750	
10:		5.875	0.563		6.125	0.568		6.125	0.586	0.586		6.125	0.610		10.750	1.558		6.625	0.696
		6.000			6.000			6.000				6.250			10.250			6.750	
		6.000			6.000			6.375				6.375			9.875			6.625	
		6.250			6.125			6.125				6.375		0.03	9.625			6.875	

FIGURE 4 CONTINUED

```
11:     6.250     0.586       6.000     0.539       6.125     0.580   0.580           6,500     0.628         9.250     1.240       6.875     0.708
        6.125                 5.875                 6.000                             6.250                   8.875                 6.750
        6.125                 5.875                 6.000                             6.375                   8.875                 6.875
        6.125                 5.875                 6.375                             6.375                   9.000                 6.625
12:     6.000     0.551       5.750     0.522       6.250     0.592   0.592           6.500     0.628         8.750     1.133       6.500     0.658
        5.875                 5.625                 6.250                             6.375                   8.875                 6.500
        6.000                 6.000                 6.125                             6.375                   8.500                 6.500
        6.000                 5.875                 6.125                             6.250                   8.250                 6.625
13:     6.125     0.586       5.875     0.557       6.000     0.562   0.562           6.375     0.616         8.375     1.022       6.500     0.658
        6.125                 5.875                 6.125                             6.250                   8.125                 6.500
        6.125                 5.875                 5.875                             6.250                   8.250                 6.500
        6.250                 6.375                 6.125                             6.375                   7.875                 6.625
14:     6.125     0.551       5.750     0.534       6.000     0.539   0.539           6.500     0.622         8.000     0.908       6.375     0.622
        6.000                 6.125                 6.000                             6.375                   7.750                 6.500
        5.875                 5.875                 5.875                             6.250                   7.625                 6.250
        5,875                 5.750                 5.750                             6.250                   7.375                 6.250
15:     6.000     0.522       5.750     0.488       5.875     0.488   0.488           6.000     0.522         7.375     0.828       6.250     0.598
        5.625                 5.500                 5.625                             5.750                   7.500                 6.125
        5.875                 5.750                 5.500                             5.750                   7.250                 6.375
        5.750                 5.500                 5.500                             5.750                   7.250                 6.125
16:     5.500     0.494       5.375     0.489       5.250     0.456   0.456           5.750     0.494         7.125     0.779       6.250     0.610
        5.625                 5.625                 5.625                             5.625                   7.125                 6.375
        5.750                 5.875                 5.500                             5.500                   7.125                 6.250
        5.750                 5.625                 5.375                             5.750                   7.125                 6.250
17:     5.750     0.545       5.625     0.522       5.500     0.450   0.450           5.750     0.494 0.04    7.000     0.905       6.375     0.616
        6.000                 5.875                 5.500                             5.625                   7.250                 6.250
        5.875                 5.625                 5.375                             5.625                   8.250                 6.250
        6.125                 6.125                 5.250                             5.625                   8.125                 6.375
18:     6.125     0.604       6.125     0.574       5.250     0.439   0.439           5.625     0.499         8.500     1.037       6.250     0.622
        6.250                 6.125                 5.375                             5.750                   8.250                 6.375
        6.375                 6.000                 5.375                             5.750                   8.000                 6.375
        6.250                 6.125                 5.375                             5.625                   8.125                 6.375
19:     6.250     0.604       6.125     0.557       5.375     0.472   0.472           5.625     0.494         8.000     0.930       6.500     0.658
        6.250                 6.125                 5.500                             5.625                   7.875                 6.500
        6.250                 5.875                 5.625                             5.750                   7.625                 6.625
        6.250                 5.875                 5.625                             5.625                   7.625                 6.500
20:     6.125     0.574       5.875     0.511       5.500     0.477   0.477           5.875     0.516         7.500     0.814       6.375     0.646
        6.000                 5.750                 5.625                             5.625                   7.250                 6.625
        6.125                 5.750                 5.625                             5.750                   7.125                 6.500
        6.125                 5.625                 5.500                             5.875                   7.250                 6.375
21:     5.875     0.545       5.625     0.472       5.500     0.466   0.466           5.750     0.528         7.125     0.773       6.500     0.616
        6.000                 5,625                 5.500                             5.875                   7.125                 6.125
        6.000                 5.375                 5.625                             5.875                   7.125                 6.250
        5.875                 5.500                 5.375                             5.875                   7.000                 6.375
22:     6.000     0.545       5.375     0.439       5.375     0.424   0.424           5.875     0.516         7.000     0.740       6.375     0.622
        5,875                 5.375                 5,125                             5.625                   7.000                 6.250
        6.000                 5.375                 5.250                             5.875                   6.875                 6.375
        5.875                 5.250                 5,250                             5.750                   6.875                 6.375
23:     6.000     0.511       5.250     0.424       5.000     0.388   0.388           5.625     0.466         6.750     0.708       6.375     0.604
        5.625                 5.250                 5.000                   0.02      5.625                   6.750                 6.250
        5.750                 5.250                 5.125                             5.375                   6.750                 6.250
        5.625                 5.250                 5.000                             5.375                   6.875                 6.125

ADF               0.489                 0.468                 0.431   0.431                     0.628                   1,199                 0.611
 RN                                                                             0.71                    1.28
SITE:H2                                                                                                                                      M3670266
```

FIGURE 5



H2
JUNE 18-23, 1981
STRATFORD, ONTARIO

FIGURE 6

INFILTRATION/INFLOW ANALYSIS OF STRATFORD, ONTARIO
PREPARED FOR THE CITY OF STRATFORD AND THE MINISTRY OF THE ENVIRONMENT
GROSS FLOW MONITORING (PATENT # 4070563)

DRY WEATHER FLOW SUMMARY

SITE	F2 17	F3 2	F4 1	G1 19	G2 28	G3 16	H2 21	H3 20	H4 25	H5 26	H6 27
DRY DAYS (SEE CALENDAR)	1,0:00- 2,23:45	1,0:00- 2,23:45	3,0:00- 3,23:45	1,0:00- 2,23:45	1,0:00- 2,23:45	1,0:00- 2,23:45	1,0:00- 2,23:45	1,0:00- 2,23:45	1,0:00- 2,23:45	1,0:00- 2,23:45	1,0:00- 2,23:45
DAILY FLOWS (MGD)	0.875	0.265	0.384	0.686	0.146	0.052	0.478	0.080	0.104	0.065	0.061
PEAK READING (MGD)	1.127	0.282	0.448	0.849	0.163	0.059	0.589	0.122	0.162	0.097	0.079
ABS. MINIMUM (MGD)	0.638	0.230	0.315	0.469	0.130	0.041	0.279	0.045	0.026	0.026	0.042
WW PRODUCTION (MGD)	0.269	0.039	0.078	0.247	0.019	0.013	0.327	0.040	0.089	0.045	0.022
[illegible] INFIL (MGD)	0.606	0.225	0.306	0.439	0.128	0.039	0.152	0.040	0.015	0.020	0.039
WWP @ MINIMUM (MGD)	0.032	0.005	0.009	0.030	0.002	0.002	0.127	0.005	0.011	0.005	0.003

WET WEATHER FLOW SUMMARY

INFLOW

INTERVAL
(SEE CALENDAR)

RAINFALL (")
PEAK READING (MGD)
I/I RATE @ PEAK (MGD)
INFL RATE @ PEAK (MGD)

RAIN INDUCED I/I (TG)
COMPENSATION (TG)

ACTUAL INFLOW (TG)

HIGH GROUNDWATER

	F2	F3	F4	G1	G2	G3	H2	H3	H4	H5	H6
INTERVAL (SEE CALENDAR)	6,0:00- 6,7:45	6,0:00- 6,7:45	6,0:00- 6,7:45	6,0:00- 6,7:45	6,0:00- 6,7:45	6,0:00- 6,7:45	6,0:00- 6,7:45	6,0:00- 6,7:45	6,0:00- 6,7:45	6,0:00- 6,7:45	6,0:00- 6,7:45
MINIMUM (MGD)	0.805	0.256	0.408	0.631	0.163	0.046	0.499	0.189	0.079	0.035	0.105
INFILTRATION (MGD)	0.773	0.252	0.398	0.601	0.161	0.044	0.372	0.184	0.068	0.029	0.103
INFIL. INCREASE (MGD)	0.167	0.027	0.093	0.162	0.033	0.005	0.220	0.144	0.053	0.009	0.064
SITE	F2	F3	F4	G1	G2	G3	H2	H3	H4	H5	H6

FIGURE 6A

INFILTRATION/INFLOW ANALYSIS OF STRATFORD, ONTARIO

PREPARED FOR THE CITY OF STRATFORD AND THE MINISTRY OF THE ENVIRONMENT
GROSS FLOW MONITORING (PATENT # 4070563)

DRY WEATHER FLOW SUMMARY

SITE	F2 17	F3 2	F4 1	G1 19	G2 28	G3 16	H2 21	H3 20	H4 25	H5 26	H6 27
DRY DAYS (SEE CALENDAR)	3,0:00- 3,23:45	3,0:00- 3,23:45	3,0:00- 3,23:45	3,0:00- 3,23:45	3,0:00- 3,23:45	3,0:00- 3,23:45	3,0:00- 3,23:45	3,0:00- 3,23:45	1,0:00- 2,23:45	3,0:00- 3,23:45	3,0:00- 3,23:45
DAILY FLOWS (MGD)	0.696	0.233	0.384	0.549	0.160	0.040	0.431	0.074	0.104	0.056	0.059
PEAK READING (MGD)	0.849	0.248	0.448	0.710	0.176	0.050	0.592	0.115	0.162	0.099	0.088
ABS. MINIMUM (MGD)	0.524	0.208	0.315	0.411	0.142	0.027	0.292	0.045	0.026	0.023	0.040
WW PRODUCTION (MGD)	0.196	0.028	0.078	0.157	0.020	0.015	0.228	0.033	0.089	0.038	0.021
LOW INFIL (MGD)	0.500	0.205	0.306	0.392	0.140	0.025	0.203	0.041	0.015	0.018	0.038
AMP @ MINIMUM (MGD)	0.023	0.003	0.009	0.019	0.002	0.002	0.089	0.004	0.011	0.005	0.003

WET WEATHER FLOW SUMMARY

INFLOW

SITE	F2	F3	F4	G1	G2	G3	H2	H3	H4	H5	H6
INTERVAL (SEE CALENDAR)	4,2:00- 4,6:45	4,2:00- 4,5:45	4,2:00- 4,5:45	4,2:00- 4,6:45	4,2:00- 4,6:45	4,2:00- 4,5:45	4,2:00- 4,6:45	4,2:00- 4,6:45		4,2:00- 4,5:45	4,2:00- 4,6:45
RAINFALL (")	0.69	0.69	0.69	0.69	0.69	0.69	0.69	0.69		0.69	0.69
PEAK READING (MGD)	2.425	0.495	0.609	2.549	1.629	0.118	1.865	0.615		0.598	0.536
I/I RATE @ PEAK (MGD)	1.654	0.287	0.294	2.138	1.474	0.085	1.561	0.563		0.574	0.490
INFL RATE @ PEAK (MGD)	1.845	0.284	0.296	2.076	1.462	0.079	1.534	0.528		0.564	0.484
RAIN INDUCED I/I (TG)	199.06	22.23	27.93	215.98	89.86	7.46	155.05	62.54		40.14	38.76
COMPENSATION (TG)	12.39	1.47	1.54	14.67	3.28	1.07	13.28	11.09		3.42	3.45
ACTUAL INFLOW (TG)	186.67	20.77	26.40	201.30	86.58	6.39	141.76	51.45		36.73	35.31

HIGH GROUNDWATER

INTERVAL
(SEE CALENDAR)

MINIMUM (MGD)
INFILTRATION (MGD)
INFIL. INCREASE (MGD)

SITE F2 F3 F4 G1 G2 G3 H2 H3 H4 H5 H6

FIGURE 7

INFILTRATION/INFLOW ANALYSIS OF STRATFORD, ONTARIO
PREPARED FOR THE CITY OF STRATFORD AND THE MINISTRY OF THE ENVIRONMENT
GROSS FLOW MONITORING (PATENT #4070563 #4211111)
RAIN EVENT: JUNE 21, 1981

FLOW ISOLATION ANALYSIS: INFLOW

MINISYSTEM	# OF MANHOLES	LINEAR FEET	INCH-MILES	CUMULATIVE TOTALS (TG)	NET PER MINISYSTEM (TG)	% OF TOTAL INFLOW	GAL/ 1000 FEET	53.6% OF TOTAL INFLOW	60.6% OF TOTAL INFLOW	70.1% OF TOTAL INFLOW	80.6% OF TOTAL INFLOW
A1	42	12013	41.97	45.4	22.0	2.7	1827.19	-	-	-	-
A2	50	11744	20.19	12.8	12.8	1.6	1092.47	-	-	-	-
A3	32	7892	21.76	9.2	9.2	1.1	1170.81	-	-	-	-
A4	65	17876	45.23	1.3	1.3	0.2	74.40	-	-	-	-
B1	35	13940	26.62	43.3	43.3	5.2	3106.89E	-	-	13940- 35	13940- 35
B2	48	13532	26.50	34.8	34.8	4.2	2575.38	-	-	-	13532- 48
E1	7	2913	24.21	593.8	103.6	12.5	35564.66	2913- 7	2913- 7	2913- 7	2913- 7
E2	92	25287	58.68	50.3	37.4	4.5	812.01	-	-	-	-
E3	32	8808	14.19	5.9	5.9	0.7	672.12	-	-	-	-
E4	84	20833	36.41				BU				
E5	86	20025	39.26	6.9	6.9	0.8	345.57	-	-	-	-
F1G1	62	18780	56.79	549.6	76.8	9.3	4087.34	18780- 62	18780- 62	18780- 62	18780- 62
F2	101	33713	80.49	186.7	139.5	16.9	4137.87	33713- 101	33713- 101	33713- 101	33713- 101
F3	71	19171	40.96	20.8	20.8	2.5	1083.41	-	-	-	-
F4	118	28532	66.67	26.4	26.4	3.2	925.28	-	-	-	-
G2	36	12958	23.52	86.6	86.6	10.5	6681.59	12958- 36	12958- 36	12958- 36	12958- 36
G3	18	5575	9.57	6.4	6.4	0.8	1146.19	-	-	-	-
H2H3	56	17831	39.20	193.2	11.2	1.4	389.79	-	-	-	-
H4	25	11031	19.20				I				
H5	16	5238	8.93	36.7	36.7	4.4	7012.22	5238- 16	5238- 16	5238- 16	5238- 16
H6	28	12144	21.77	35.3	35.3	4.3	2907.61	-	-	12144- 28	12144- 28
H7	51	18069	34.12	109.9	51.6	6.2	2854.06	-	-	-	18069- 51
H8	48	16894	25.22	58.3	58.3	7.1	3453.89	-	16894- 48	16894- 48	16894- 48
TOTALS	1203	354799	781.46		827.0	100.0		73602- 222	90496- 270	116580- 333	148181- 432

NOTES:

'I' INDICATES A MONITOR SITE WHERE INFLOW/INFILTRATION COULD NOT BE MEASURED. THE LINEAR FOOTAGE AND INCH-MILE FIGURES HAVE BEEN ADDED TO THOSE OF THE NEXT DOWNSTREAM SITE FOR ANALYSIS (SEE 'PROJECT NOTES')

'E' INDICATES A MONITOR SIRE WHERE THE INFLOW/INFILTRATION FIGURE HAS BEEN ESTIMATED BASED ON CALCULATIONS AT COMPARISON SITES (SEE 'PROJECT NOTES')

'BU' INDICATES A MONITOR SITE WHERE INFLOW/INFILTRATION COULD NOT BE MEASURED DUE TO BACKUP CONDITIONS. THE LINEAR FOOTAGE AND INCH-MILE FIGURES HAVE BEEN ADDED TO THOSE OF THE NEXT DOWNSTREAM SITE FOR ANALYSIS (SEE'PROJECT NOTES')

FIGURE 8

INFILTRATION/INFLOW ANALYSIS OF STRATFORD, ONTARIO
PREPARED FOR THE CITY OF STRATFORD AND THE MINISTRY OF THE ENVIRONMENT
GROSS FLOW MONITORING (PATENT #4070563 #4211111)
RAIN EVENT: JUNE 21, 1981

FLOW ISOLATION ANALYSIS: INFILTRATION

MINISYSTEM	# OF MANHOLES	LINEAR FEET	INCH-MILES	CUMULATIVE TOTALS (MGD)	NET PER MINISYSTEM (MGD)	% OF TOTAL INFIL.	GAL/ INCH-MILE /DAY	60.4% OF TOTAL INFIL.	72.9% OF TOTAL INFIL.	83.8% OF TOTAL INFIL.
A1	42	12013	41.97	0.325	0.032	0.9	762.45	-	-	-
A2	50	11744	20.19	0.063	0.063	1.8	3120.36	-	-	-
A3	32	7892	21.76	0.154	0.154	4.3	7077.21	7892- 32	7892- 32	7892- 32
A4	65	17876	45.23	0.076	0.076	2.1	1680.30	-	-	-
B1	35	13940	26.62	0.230	0.230	6.4	8640.12E	13940- 35	13940- 35	13940- 35
B2	48	13532	26.50	0.200	0.200	5.6	7547.17E	13532- 48	13532- 48	13532- 48
E1	7	2913	24.21	2.508	0.768	21.4	31722.43	2913- 7	2913- 7	2913- 7
E2	92	25287	58.68	0.423	0.324	9.0	3407.30	-	-	46120- 176
E3	32	8808	14.19	0.023	0.023	0.6	1620.86	-	-	-
E4	84	20833	36.41				BU			
E5	86	20025	39.26	0.076	0.076	2.1	1935.81	-	-	-
F1G1	62	18780	56.79	1.635	0.101	2.8	1778.48	-	-	-
F2	101	33713	80.49	0.773	0.123	3.4	1528.14	-	-	-
F3	71	19171	40.96	0.252	0.252	7.0	6152.34	19171- 71	19171- 71	19171- 71
F4	118	28532	66.67	0.398	0.398	11.1	5969.70	28532- 118	28532- 118	28532- 118
G2	36	12958	23.52	0.161	0.161	4.5	6845.24	12958- 36	12958- 36	12958- 36
G3	18	5575	9.57	0.044	0.044	1.2	4597.70	-	5575- 18	5575- 18
H2H3	56	17831	39.20	0.556	0.162	4.5	4132.66	-	17831- 56	17831- 56
H4	25	11031	19.20	0.068	0.068	1.9	3541.67	-	-	11031- 25
H5	16	5238	8.93	0.029	0.029	0.8	3247.48	-	-	-
H6	28	12144	21.77	0.103	0.103	2.9	4731.28	-	12144- 28	12144- 28
H7	51	18069	34.12	0.194	0.057	1.6	1670.57	-	-	-
H8	48	16894	25.22	0.137	0.137	3.8	5432.20	-	16894- 48	16894- 48
TOTALS	1203	354799	781.46		3.581	100.0		98938- 347	151382- 497	208533- 698

NOTES:

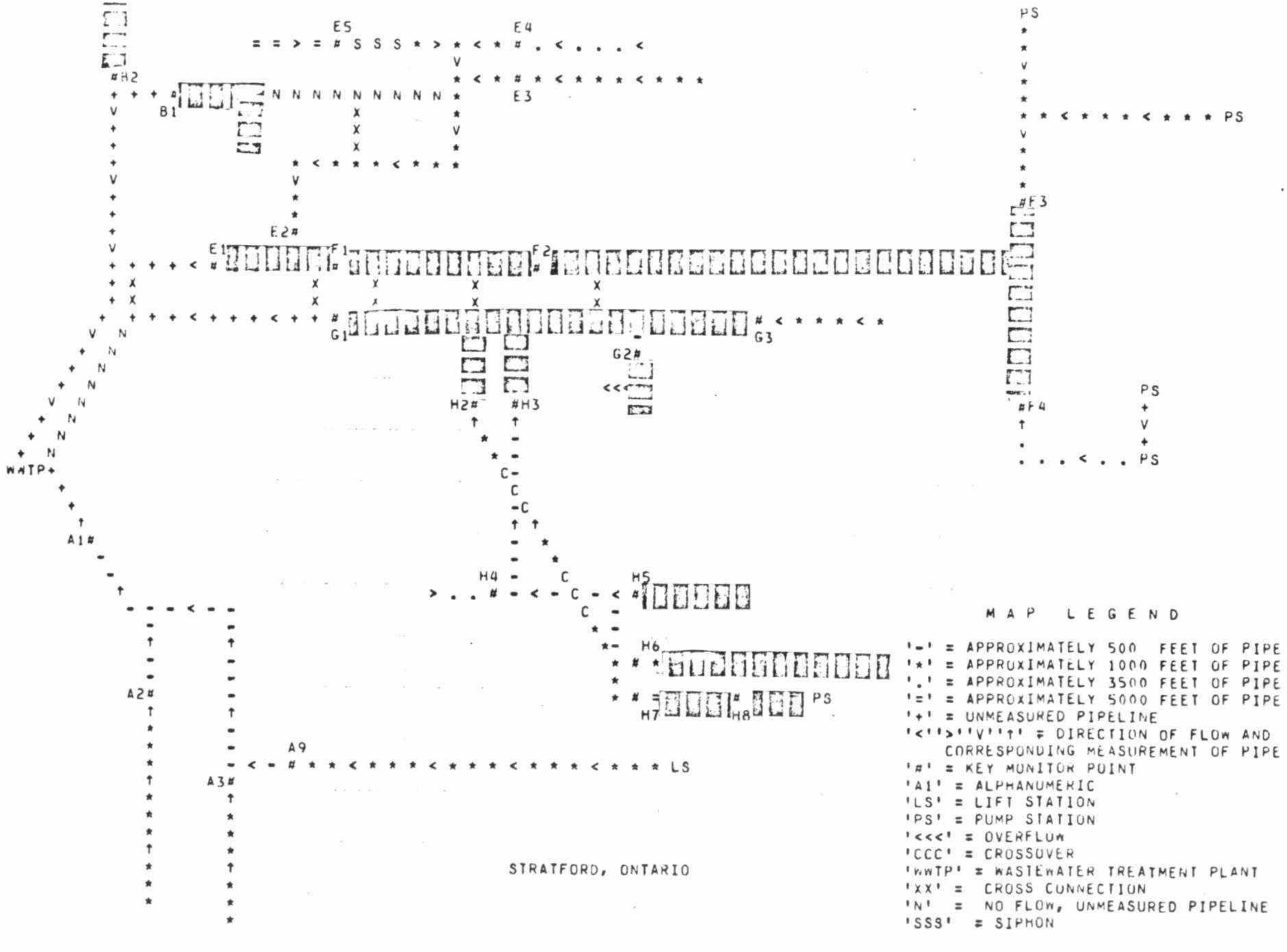
'I' INDICATES A MONITOR SITE WHERE INFLOW/INFILTRATION COULD NOT BE MEASURED. THE LINEAR FOOTAGE AND INCH-MILE FIGURES HAVE BEEN ADDED TO THOSE OF THE NEXT DOWNSTREAM SITE FOR ANALYSIS (SEE 'PROJECT NOTES')

'E' INDICATES A MONITOR SITE WHERE THE INFLOW/INFILTRATION FIGURE HAS BEEN ESTIMATED BASED ON CALCULATIONS AT COMPARISON SITES (SEE 'PROJECT NOTES')

'BU' INDICATES A MONITOR SITE WHERE INFLOW/INFILTRATION COULD NOT BE MEASURED DUE TO BACKUP CONDITIONS. THE LINEAR FOOTAGE AND INCH-MILE FIGURES HAVE BEEN ADDED TO THOSE OF THE NEXT DOWNSTREAM SITE FOR ANALYSIS (SEE'PROJECT NOTES')

FIGURE 9A

INFLOW

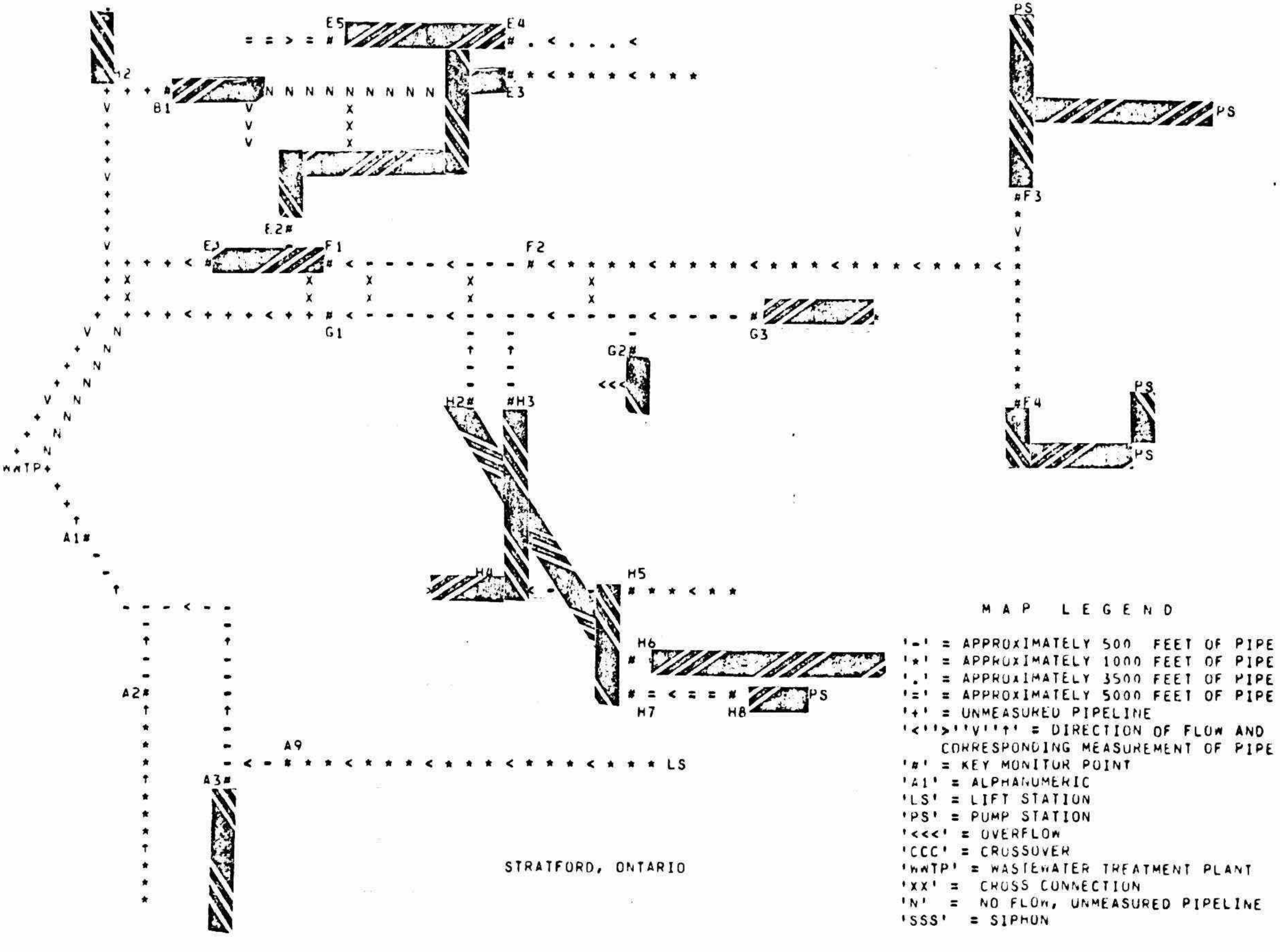


M A P   L E G E N D

'-' = APPROXIMATELY 500 FEET OF PIPE
'*' = APPROXIMATELY 1000 FEET OF PIPE
'.' = APPROXIMATELY 3500 FEET OF PIPE
'=' = APPROXIMATELY 5000 FEET OF PIPE
'+' = UNMEASURED PIPELINE
'<''>''V''↑' = DIRECTION OF FLOW AND CORRESPONDING MEASUREMENT OF PIPE
'#' = KEY MONITOR POINT
'A1' = ALPHANUMERIC
'LS' = LIFT STATION
'PS' = PUMP STATION
'<<<' = OVERFLOW
'CCC' = CROSSOVER
'WWTP' = WASTEWATER TREATMENT PLANT
'XX' = CROSS CONNECTION
'N' = NO FLOW, UNMEASURED PIPELINE
'SSS' = SIPHON

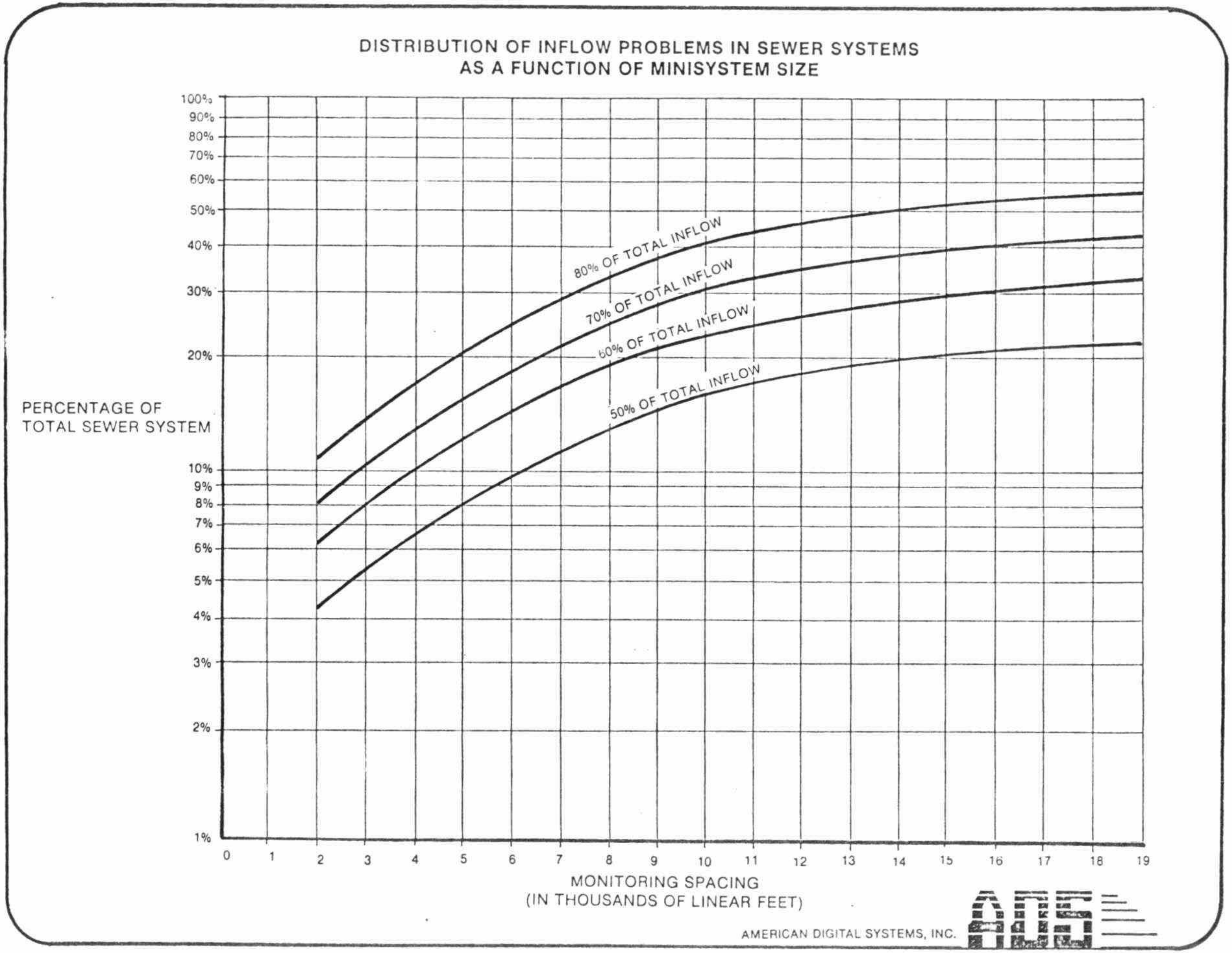
FIGURE 9B

INFILTRATION


E5
E4
E3
B1
E2
E1
F1
F2
F3
F4
PS
G1
G2
G3
H2
H3
H4
H5
H6
H7
H8
WWTP
A1
A2
A3
A9
LS
M A P   L E G E N D
'-' = APPROXIMATELY 500 FEET OF PIPE
'*' = APPROXIMATELY 1000 FEET OF PIPE
'.' = APPROXIMATELY 3500 FEET OF PIPE
'=' = APPROXIMATELY 5000 FEET OF PIPE
'+' = UNMEASURED PIPELINE
'<''>''V''↑' = DIRECTION OF FLOW AND CORRESPONDING MEASUREMENT OF PIPE
'#' = KEY MONITOR POINT
'A1' = ALPHANUMERIC
'LS' = LIFT STATION
'PS' = PUMP STATION
'<<<' = OVERFLOW
'CCC' = CROSSOVER
'WWTP' = WASTEWATER TREATMENT PLANT
'XX' = CROSS CONNECTION
'N' = NO FLOW, UNMEASURED PIPELINE
'SSS' = SIPHON

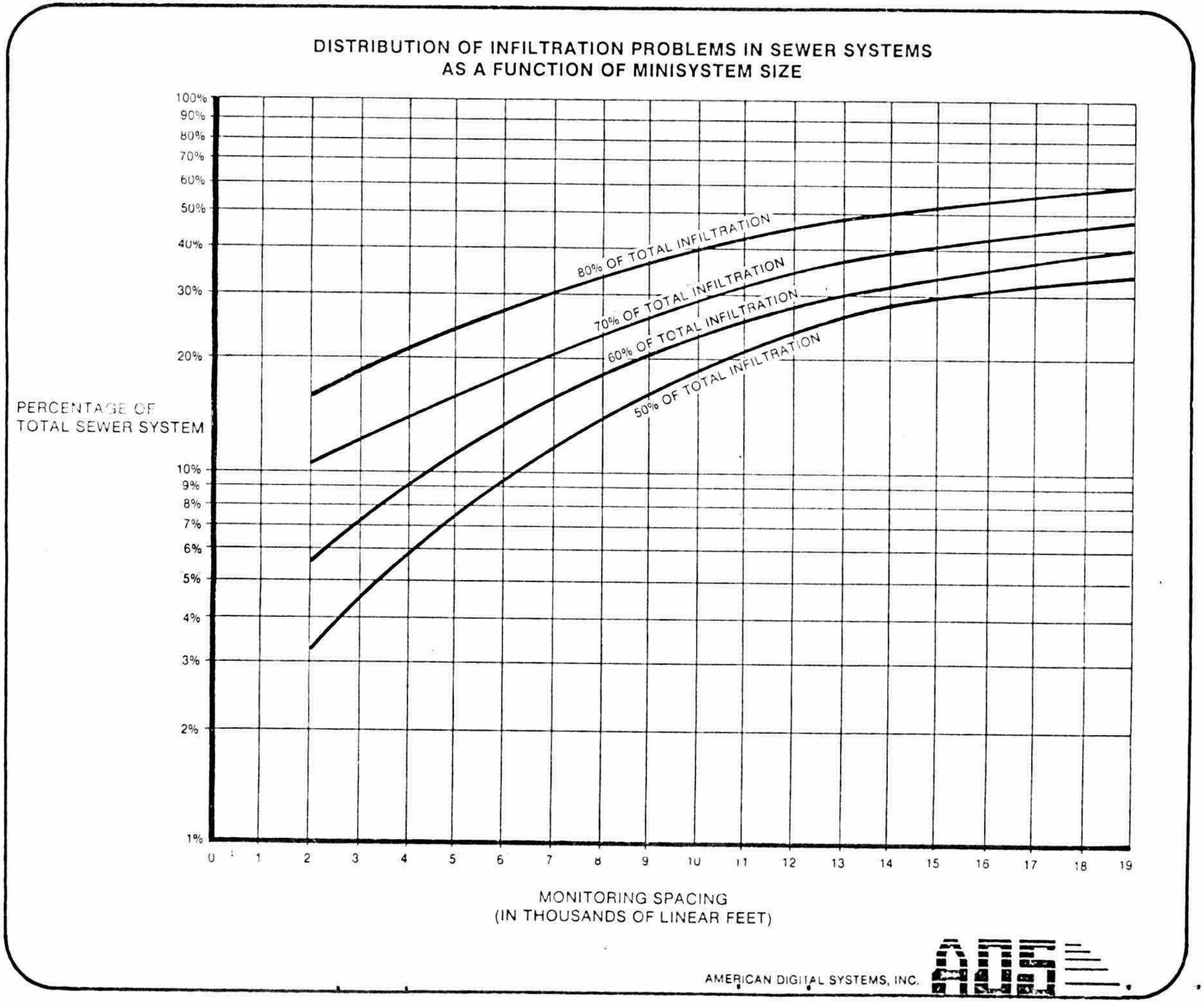
STRATFORD, ONTARIO

FIGURE 10



DISTRIBUTION OF INFLOW PROBLEMS IN SEWER SYSTEMS
AS A FUNCTION OF MINISYSTEM SIZE

FIGURE 11



DISTRIBUTION OF INFILTRATION PROBLEMS IN SEWER SYSTEMS AS A FUNCTION OF MINISYSTEM SIZE

FIGURE 12

ESTIMATED SAVINGS FROM INTENSIVE FLOW MONITORING PROGRAM

	COST WITHOUT INTENSIVE FLOW MONITORING PROGRAM	COST WITH INTENSIVE FLOW MONITORING PROGRAM	SAVINGS
SMOKE TESTING	$105,000	$26,000	$ 79,000
TELEVISION INSPECTION	236,000	60,000	176,000
TOTAL TESTING COSTS	341,000	86,000	255,000
COSTS OF COMPREHENSIVE FLOW MONITORING PROGRAM, INCLUDING ENGINEERING			(-72,000)
NET SAVINGS			$183,000

PROCESS MODIFICATION ALTERNATIVES - AN OVERVIEW

N. W. Schmidtke, Ph.D., P. Eng.[1]

INTRODUCTION

One of the most neglected areas in environmental process engineering concerns the upgrading of existing waste treatment plants. It is by far easier to duplicate rather than upgrade existing unit operations and processes. Upgrading requires an in-depth knowledge and understanding of process fundamentals.

There are a number of questions which immediately come to mind when we speak of upgrading; the major one being; 'Why should we upgrade an existing plant?' There are three specific reasons:

1. to meet more stringent effluent criteria;
2. to increase hydraulic and/or organic loading capacity;
3. to improve poor process performance due to improper design and/or operation.

Another question then arises: 'What do we upgrade?' This of course depends on the circumstances, but many times only a part of the total plant may be all that needs to be upgraded. 'How do we upgrade?' This is a function of our objective and the structural, as well as non-structural, aspects of the plant under consideration.

To tackle the upgrading problem requires a number of familiarization prerequisites such as:

1. ascertain the efficiency of treatment;
2. determine the normal operating and maintenance procedures followed at the plant;
3. examine the condition of the structures;
4. determine the condition of the process hardware;
5. find out what the staffing pattern for the plant is and what operator skill level exists.

---

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Environmental Engineering Specialists
178 Louisa Street
Kitchener, Ontario
N2H 5M5

Having done this plant records as flow charts, operating logs and all available lab data must be reviewed. It is also important to be familiar with:

1. the sampling and flow measurement techniques followed at the plant;
2. the flow measuring device calibration schedule;

With this knowledge a feel for the available data is gained, and this in turn allows one to judge its reliability.

UPGRADING TO MEET MORE STRINGENT EFFLUENT CRITERIA

Tighter effluent criteria dictates that a plant must produce a better effluent than before. There are a number of possible tighter effluent criteria which might have to be met. Examples are:

1. lower BOD_5 and SS;
2. maintenance of a minimum effluent DO;
3. more stringent effluent disinfection;
4. low phosphorus levels;
5. partial or complete, year-round or seasonal oxidation of ammonia nitrogen;
6. complete absence of nitrogen.

However, before embarking on any upgrading scheme, a number of points must be kept in mind:

1. maximize existing unit operations;
2. evaluate all alternatives;
3. upgrading will produce more sludge;
4. the existing sludge handling and disposal practice may have to be changed.

UPGRADING TO RELIEVE HYDRAULIC AND ORGANIC OVERLOADS

There is little doubt that process performance is adversely affected by overloads. As well, different upgrading approaches must be followed if on one hand an increase in hydraulic load without a comensurate increase in organic load occurs as opposed to an increase in both hydraulic and organic loading.

Causes of hydraulic overloading can, in most instances, be attributed to infiltration and the existance of combined sewers. Organic overloading is most frequently caused by industrial contributions. This effect is of course, most pronounced in small communities. When both hydraulic and organic overloading conditions are experienced, they can principally be attributed to rapid expansion of the community and/or rapid industrialization. Experience has shown that one should base the industrial organic contribution on a COD rather than BOD population equivalent.

In order to assess the severity of the overloading problem, a data base is required. Here, existing plant data on flows and influent concentrations is needed. Examination of this data will give information on the magnitude, frequency and duration of both peak flows and constituent concentrations. This information can then be used to great advantage in making future loading projections. Another important point to keep in mind is to look for weekly or seasonal load variations. This in turn may identify that only certain process components need be upgraded.

The major impact of hydraulic overloads manifests itself in the decreased solids capture ability of both primary and secondary clarifiers. Plant flow data coupled with connected population data and water useage data will assist in the identification of whether an inflow/infiltration problem exists. If so, then every effort to reduce this problem should be made.

A number of causes for organic over-loading of treatment plants have been identified, and may consist of:

1. industrial loads;
2. increased volumes of septage to the plant;
3. poorly operating or overloaded sludge processing operations and resulting sludge recycle streams such as:
    - thickeners
    - digesters
    - elutriation tanks
    - vacuum filters

The ultimate result of organic overloading could very well be complete process failure.

Certain remedies for organic overloads caused by industry are available and may consist of:

- pre-treatment
- equalization
- expansion of certain process components

An overload condition caused by a high influent BOD has little impact on the primary stage other than causing odour if the waste is highly biodegradable. It is a different situation though for the biological stage. Here oxygen supply, sludge return capacity, sludge wasteage and of course additional sludge production all impact on overall process performance.

High influent suspended solids will result in increased primary and, to a lesser extent, secondary sludge production. This may well result in exceeding the available capacity of existing thickeners, digesters and various sludge dewatering equipment.

In order to assess the overloading problem, one must be able to compare design criteria to operating data, but this of course, can only be done if good plant records are kept. Armed with this information, the degree of overload can be determined and the appropriate course of corrective action can be charted.

Generally speaking, the use of mass balances using phosphorus and suspended solids are excellent tools to determine the validity of both flow measurement and analytical data. The solids balance can be used to good advantage to identify the source and magnitude of excessive in-plant recycle loads. Phosphorus balances, because phosphorus does not break down to a gaseous form during treatment, can also be used. Where phosphorus removal in a treatment plant is practiced, this of course is not possible.

Plants in small communities can also suffer from an organic overloading problem caused by the addition of septage. Experience has shown that if the septage to total plant flow when based on a kg BOD_5 d^{-1} basis is small, the septage should be put into a separate, aerated basin and bled into the plant at a controlled rate. If the ratio is high or if the septage is toxic, then another strategy is required. This may well consist of separate treatment such as:

1. aerobic digestion — dewatering
2. anaerobic digestion -- dewatering
3. the above -- bleeding into the existing plant

At times, one may also be faced with the problem of having to upgrade a plant to improve on its original design and/or operation. A plant may be well designed but not functioning properly because of inadequate operator training, inadequate operating staffing or lack of financial support. The converse may also be true. A poorly designed plant may function beyond design expectations because of the extraordinary efforts put forth by the plant operations staff.

In the assessment of plant design adequacy the plant operator has a wealth of information. He is thus the best information source, and will be quick to identify major design deficiencies. A summary of some of the most commonly identified shortcomings are as follows:

1. inadequate stand-by equipment;
2. fixed speed units where variable speed is essential, i.e., - raw wastewater pumps
   - recycle pumps
   - air blowers
3. poor hydraulic and solids distribution among identical units operating in parallel;
4. insufficient or inflexible return and sludge wasting pump capacity;
5. inability of instrumentation and equipment to operate at low flows and loads in early plant life;
6. lack of dewatering facilities to permit rapid emptying of process tanks;
7. lack of disinfection system flexibility if required to control odour and/or sludge bulking;
8. inadequate electrical and hydraulic capacity to permit stand-by pumps and air blowers to operate in parallel;
9. inadequate laboratory facilities;
10. lack of assessibility to key mechanical equipment for routine maintenance or repair.

Process design engineers should pay particular attention to the aforenoted points since their consideration and inclusion in the initial design will avoid future problems and embarrassment. From the process design point of view, one of the most common deficiencies is a plant design based on average flow, BOD_5 and SS mass loadings and no recognition of the fact that the plant will be subjected to peak loads which may be 2-3 times the average daily load in many cases. On the other hand, it must also be recognized that peak flow, peak BOD_5 and SS mass loadings do not occur simultaneously. Thus a design based on the occurence of these peak events simultaneously may well result in excess plant capacity.

A good rule to follow is that no physical plant upgrading of any sort should occur until one is certain that the plant is operated to yield its maximum efficiency. The proper tool for good process control is frequent and accurate sampling as well as analytical analyses. For instance, the location of solids sampling is critical. If the solids level is high, such as in the return sludge, solids have a tendency to flow near the bottom part of the pipe and thus it is more difficult to obtain a representative sample. For proper plant operation laboratory support, plant equipment, staff and funds must also be adequate.

UPGRADING TECHNIQUES

The following are a number of upgrading techniques which have been successfully used in the past and should be considered as candidates for application:

Event	Upgrading Techniques
smoothing of flow	equalization
increased SS removal efficiency	chemical coagulation use of polyelectrolytes use of novel settling devices
modifications to activated sludge	step aeration contact stabilization complete mix replace air with oxygen

increased biological capacity for overloaded plant	provide additional treatment ahead of existing biological plant
increased benefit of DO in receiving waters	nitrify effluent, especially in summer.

Should more stringent effluent criteria be required, then technologies such as the following should be considered:

1. Effluent polishing;
2. Aerobic & facultative lagoons;
3. Microscreens;
4. Multi-media filtration;
5. Chemical coagulation;
6. Biological nitrification and/or denitrification;
7. Activated carbon treatment.

Aside from techniques for upgrading the various liquid processing steps, opportunities for upgrading the solids handling portion as it pertains to volume reduction and stabilization must not be lost. In recent years a number of techniques addressing this subject have been exploited to good advantage. They are identified as:

1. High-rate anaerobic digestion;
2. Aerobic digestion;
3. Thickening of waste activated sludge:
    - air flotation
    - centrifugal thickening
4. Polyelectrolyte addition to improve thickening and dewatering;
5. Heat treatment processes for sludge stabilization.

When faced with the problem of having to upgrade a treatment plant, one must also keep the following points in mind:

1. the physical condition of the plant under consideration;
2. the length of time before a major plant expansion is required;
3. the time required for the implementation of various upgrading techniques;
4. compatability of upgrading with the future expansion;
5. the financial assistance available to the community;
6. the cost of upgrading, including O&M and capital.

In any event, an economic comparison between all alternatives is essential. It would be appropriate at this time then to examine a few of the upgrading techniques which have been used successfully. One relative low cost, and hence attractive upgrading technique for an overloaded plant, is the use of flow equalization. It should be noted though, that infiltration due to a high groundwater table can seldom be successfully equalized. As a 'band-aid' solution it is sometimes possible to use existing large interceptors for storage.

Flow Equalization

Flow equalization will achieve a number of objectives such as:

1. dampen diurnal flow variations;
2. dampen both concentration and mass flow of various wastewater constituents;

As a consequence, flow equalization will:

1. significantly improve the performance of an existing treatment facility; and
2. result in the size reduction of downstream process components when a new plant is being designed.

There are two types of flow equalization concepts as shown in Figure 1.:

1. In-line: where everything passes through the equalizatank(s) and thus results in a significant dampening of both flow variations as well as wastewater constituents.

2. Off-line: here only the flow above the average flow passes to the equalization tank(s). While one will be able to reduce the overall pumping costs, the disbenefit manifests itself in less effective concentration damping.

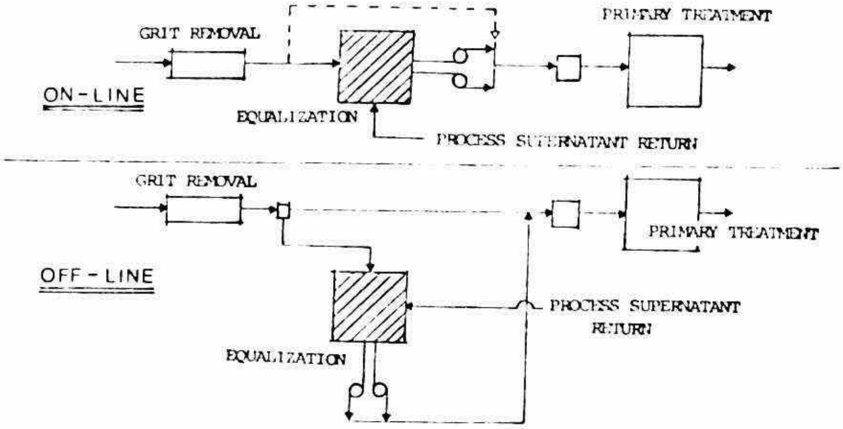


FIGURE 1 TYPES OF EQUALIZATION ALTERNATIVES

Flow equalization is most effective when the opportunity to design the flow equalization basins exists, rather than the modification of available reactors. To be most effective, flow equalization facilities should be located near the head end of treatment works, but downstream of such units as:

- bar screens
- comminutors
- grit chambers

Adequate mixing and aeration must be provided in order to avoid odours and solids settling.

Flow equalization has also an immediate impact on primary settling. It not only results in a reduction of the peak overflow rate and a more uniform effluent, but allows the design engineer to base his design on equalized flow rate rather than peak rates. One example taken from the literature does illustrate the effect of flow equalization by increased solids capture as follows:

	Normal mgL^{-1}	Equalized mgL^{-1}
primary influent SS	137	128
primary effluent SS	105	68
% removal	23	47

In conjunction with improving settling tank performance by flow equalization techniques one should not overlook the opportunity of significantly improving primary settling by using pre-aeration to pre-flocculate the suspended solids. However, if centrifugal pumping to the primaries is used, much of this advantage is lost due to shearing of the floc.

Equalization also has a significant positive impact on the biological treatment stage. In order to maximize this advantage though, in-line flow equalization should be practiced. This results in a number of beneficial effects such as:

1. protection of process from upset or failure;
2. improved process performance where aeration equipment is marginal;
3. pH control;
4. flow smoothing, with resultant more stable solids loading to the secondary clarifier and therefore improved final settling;
5. the MLSS concentration in the aeration tank can also be increased. This results in a decreased F/M and thus increased SRT. Cascading process effects would be an increased level of nitrification and a decrease in the total amount of sludge produced. A prerequisite however, is that an adequate air supply is available;
6. the diurnal variations in the sludge blanket level will be decreased. This reduces the potential for solids being drawn over the weir.

In addition, a number of other, miscellaneous benefits can be identified:

1. Where chemical coagulation and precipitation using metal salts is practiced, reduced amounts of precipitant will be required because coagulant dosage is approximately proportional to the material to be precipitated.

2. Chemical feed control is improved. This results in greater process reliability.

3. Process instrumentation complexity is reduced, with concommitant capital cost reductions, fewer breakdowns and more appreciative plant operators.

4. Reduction in filter media surface area.

5. Enhanced filter performance.

6. A more uniform solids loading to the filter(s) and thus more uniform filtration cycles, all due to a more constant filter loading rate.

7. An excellent return point for recycle streams such as:
   - digester supernatant
   - sludge dewatering equipment filtrate
   - filter backwash.

8. Claims for a 10-20% in BOD_5 reduction have also been made.

The effect of flow equalization is summarized by Figure 2. Here curves 1 and 2 show the diurnal variations in flow and mass BOD_5 loading before flow equalization. Curves 3 and 4 illustrate the same information after flow equalization. The changes are quite dramatic.

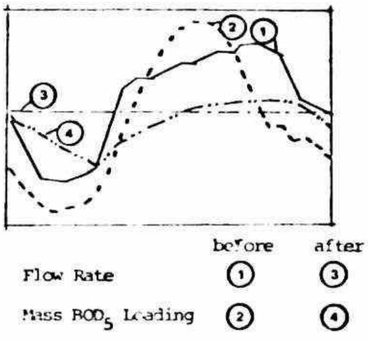


FIGURE 2 EFFECT OF FLOW EQUALIZATION

Raw wastewater flow and BOD_5 variations before and after equalization can also be shown on a flow ratio basis as follows:

Flow Ratio	Before Equalization	After Equalization
peak : average	2.0	1.2
minimum : average	0.14	0.61
peak : minimum	14.6	2.0

The costs for flow equalization are highly variable as shown in Table 1 where some relative costs for treatment plant sizes from 3800 - 38000 m^3d^{-1} are given. The flow equalization basins are either of earthen or concrete construction, with or without pumping. As well, the earthen basins have a plastic liner and floating aerators. The concrete tanks have diffused aeration. In both examples, where pumping is required, the cost of a separate pumping station is included. All costs are relative to pumped flow equalization to an earthen basin. The index is 1.0.

TABLE 1 RELATIVE COSTS FOR DIFFERENT FLOW EQUALIZATION ALTERNATIVES

PLANT SIZE	STORAGE VOLUME	RELATIVE COSTS			
		EARTHEN		CONCRETE	
m^3h^{-1}	m^3	PUMP	NO PUMP	PUMP	NO PUMP
158	1200	1.00	0.58	1.41	1.00
475	3300	1.37	0.68	2.69	2.00
1580	9100	2.56	1.08	6.28	4.80
		- plastic liner - floating aerators - separate pumping station		- diffused aeration - separate pumping station	

Techniques for Upgrading Activated Sludge Plants

It is possible and highly desirable to design an activated sludge plant to be operated in various process modes as conditions require. It will challenge process design engineers to identify designs which will allow for increased future loading conditions or improved effluent specifications at little extra expense. It is indeed possible to design a biological process so that it can be operated in several of the following modes:

1. conventional activated sludge;
2. step aeration;
3. contact stabilization;
4. complete mix;
5. modified aeration;
6. two stage;
7. pure oxygen;

Since process results are almost equal (except when nitrification is required) the air requirements will also be approximately identical. Exceptions are modified aeration (lower) and pure oxygen (higher). The oxygen and air requirements for these various process alternative are summarized in Table 2.

TABLE 2 OXYGEN/AIR REQUIREMENTS FOR VARIOUS PROCESS ALTERNATIVES

PROCESS	kg O_2 kg^{-1} BOD$_r$	std. m^3 kg^{-1} BOD$_r$
CONVENTIONAL	0.8 - 1.1	50 - 100
STEP AERATION	0.7 - 1.0	50 - 75
CONTACT STABILIZATION	0.7 - 1.0	50 - 75
COMPLETE MIX	0.7 - 1.0	50 - 75
MODIFIED AERATION	0.4 - 0.6	25 - 50
PURE OXYGEN	0.8 - 1.4	—

An excellent operational tool for the plant operator is of course the sludge recycle and sludge wastage rate.

The process design engineer must place particular emphasis on providing the maximum possible degree of flexibility for the operator when designing and selecting this equipment. If done correctly, the management of the sludge inventory, maintenance of optimum loading conditions and control of sludge settling characteristics all will ensure that the plant operator has the tools for turning out a superior effluent.

Sludge Recycle Control Techniques

Basically three sludge recycle control techniques are available:

1. automatically varying the recycle flow to maintain a set relationship to the influent waste load.
2. set the recycle flow to a constant rate based on the average daily flow.
3. control the sludge recycle pumps with sludge blanket sensor(s) based on a predetermined blanket level.

Another concept which process design engineers should keep in mind, is that of firm sludge recycle capacity (FSRC). This is defined as the available sludge recycle pumping capacity with the largest pump out of service.

Experience has shown that the following FSRC should be supplied:

1. for conventional activated sludge and step aeration -
   FSRC = 0.50 x average daily flow
2. for contact stabilization and complete mix -
   FSRC = 1.0 x average daily flow

SELECTION OF PROCESS UPGRADING ALTERNATIVES

Two questions arise in this regard:

1. How does one decide what process upgrading alternatives should be explore?
2. How does one determine process design parameters?

The second question is easier to answer. For process design parameter generation, a conventional two step procedure is followed:

1. batch studies for the treatability evaluation
2. continuous studies for design data generation, i.e., BOD_5 removal rate, oxygen requirements and sludge production.

The first question is a function of what the existing process hardware consists of. For example, upgrading alternatives for organically overloaded conventional activated sludge plants may consist of processes such as:

1. Step aeration ✓
2. Contact stabilization
3. Complete mix
4. Pure oxygen

Experience has shown alternatives 1 and 2 to be the lowest capital cost upgrading solutions. From this information the process design engineer should note that process flexibility to operate the plant in both modes 1 and 2 can be achieved by sizing the influent step aeration piping so that the entire flow could be introduced into the last compartment. This then would be the contact stabilization mode. Additional aeration capacity and structural modifications will also be required.

Figures 3, 4 and 5 illustrate various possible process modifications. In Figure 3 a conventional activated sludge plant can be modified to a step aeration plant by adding additional piping for influent flow distribution as shown. Significant structural changes to the aeration tank and minor piping modifications are required to change a conventional activated sludge process to a contact stabilization process (Figure 4). Transforming a contact stabilization process to a complete mix process is shown in Figure 5. As for the case of modification to step aeration, major changes are called for in the influent flow distribution arrangement.

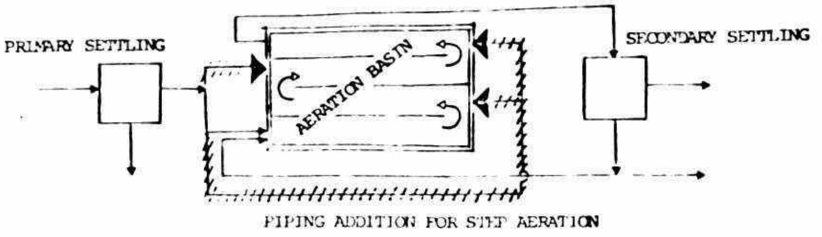


FIGURE 3 CAS --▶ STEP AERATION

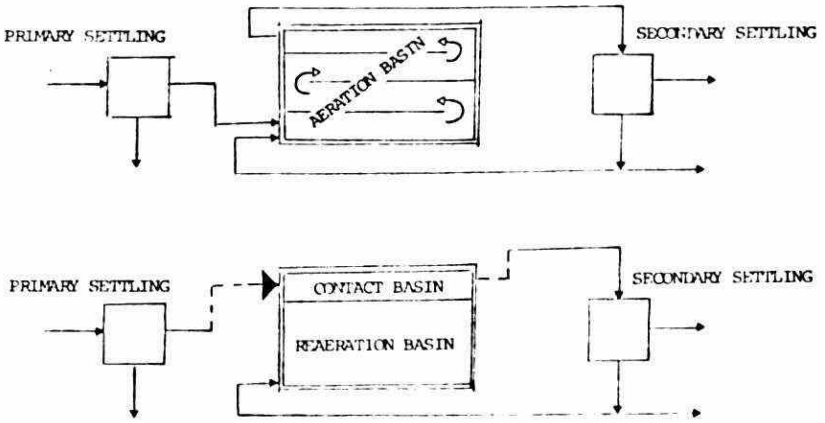


FIGURE 4 CAS --▶ CONTACT STABILIZATION

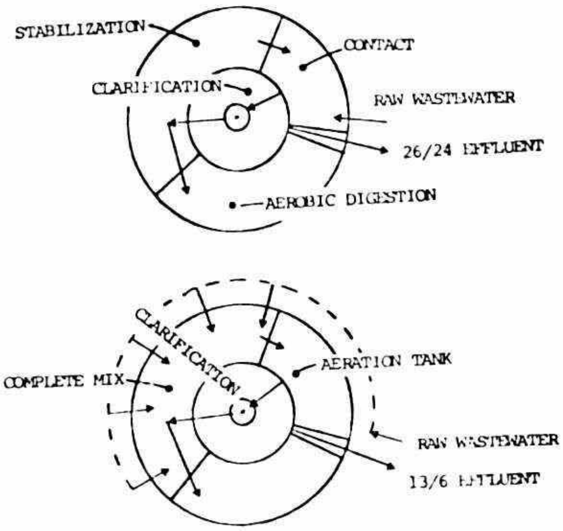


FIGURE 5 CONTACT STABILIZATION -▶ COMPLETE MIX

Effect of Process Changes

In the case where the conventional activated sludge process was upgraded to step aeration (Figure 3), a signficant increase in plant hydraulic capacity from 28,000 m^3 hr^{1-} to 39,500 m^3 hr^{1-} was achieved at a cost (1975) of \$6.60 m^3. Effluent characteristcs remained unchanged.

Another example concerns the conversion of a conventional activated sludge plant to contact stabilization similar to that shown in Figure 4. Both hydraulic plant capacity as well as process performance were increased as shown in Table 3.

TABLE 3 UPGRADING EFFECT -- CAS -- CONTACT STABILIZATION

PARAMETER		BEFORE UPGRADING		AFTER UPGRADING	
	m^3h^{-1}	1550		2460	
		IN	OUT	IN	OUT
BOD_5	mgL^{-1}	249	91	358	21
SS	mgL^{-1}	243	73	247	21

TABLE 4 UPGRADING ALTERNATIVES FOR IMPROVED ORGANIC REMOVAL EFFICIENCY

EXISTING PROCESS = ACTIVATED SLUDGE

ADDITION PRECEDING EXISTING PROCESS	ADDITION FOLLOWING EXISTING PROCESS	INCREMENTAL BOD REMOVAL ACROSS ADDED OR MODIFIED PROCESS
		%
ROUGHING T. FILTER		20 - 40
CHEM. ADD. TO PRIMARY CLARIFIER		30 - 50
	2nd STAGE A.S.	30 - 70
	POLISHING LAGOON	30 - 60
	MULTIMEDIA FILTERS	50 - 80
	MICROSCREENS	30 - 80
	ACTIVATED CARBON	60 - 80

Upgrading Alternatives for Improved Organic Removal Effiency

A variety of process modification alternatives to improve organic removal efficiency are available. In the illustrations shown in Table 4, the existing process is conventional activated sludge. As indicated, the incremental BOD removal across an added or modified process can vary anywhere from 20 to 80%.

Upgrading for Nutrient Control

The Province of Ontario has successfully completed an extensive program of plant upgrading to achieve effluent total phosphorus levels of 1.0 mgL^{-1}. This was in response to both the Canada/Ontario and the Canada/US Agreement on Water Quality for the Great Lakes. A number of workshops and technology transfer seminars reporting on these activities in detail have been held in the past 8 years. Further, site-specific controls for both phosphorus and nitrogen are continuously being implemented in Ontario as the situation demands. Other Provinces in Canada also have site specific requirements for nutrient removal.

Upgrading for nutrient control in the case of phosphorus can be summarized by the following points:

1. phosphorus precipitation to 1.0 mgL^{-1} is practiced with metal salt addition chiefly to the aeration tank.
2. precipitant addition at various other locations is possible.
3. the use of a multi-addition point concept should not be overlooked.
4. an increased solids loading to the clarifiers will be experienced
5. as a consequence of precipitant addition there will be a decrease in the volatile solids concentration in the sludge.
6. in many instances increased solids handling capacity, return sludge pumping capacity and processing will be required.
7. experience has shown that sludge volumes increase by 60% and 35% at primary and secondary plants respectively.

If upgrading for nitrogen control is to be practiced, then for nitrification the following points should be borne in mind:

1. the process is temperature, SRT dependent
2. since the oxidation of ammonia is linear, reactors should be designed for plug-flow
3. for added process flexibility, a number of compartmentalized reactors should be provided
4. must provide adequate aeration capacity to oxidize ammonia nigrogen
5. air requirements can very widely.

If the next step of nitrogen removal is taken, then biological denitrification technology offers in most instances, the most cost effective solution. Other process alternatives are ammonia stripping, chlorination and ion exchange using clinoptilolite. Details concerning denitrification technology is beyond the scope of this paper.

Impact of Upgrading Clarifiers

Since clarification is the most economical way to remove suspended and colloidal pollutants, every effort should be made to improve the clarification process before considering additional primary or secondary clarification facilities. Effective SS removal from secondary clarifiers has additional process benefits of:

1. more efficient disinfection
2. reduction in frequency of chlorine contact tank cleaning
3. production of an aesthetically pleasing effluent

As far as primary clarifiers are concerned, a number of advantages of improved performance are summarized as follows:

1. increased SS/BOD_5 removal at same flow
2. same SS/BOD_5 removal at increased flow
3. increased quantity of primary sludge which is more rapidly thickened and dewatered then secondary sludge
4. lower secondary sludge production
5. decreased organic loading to the biological stage

The impact of improved liquid solid separation through clarification on following treatment components is considerable when considering that every additional mg of VSS captured results in an approximate reduction of 0.6 mg BOD_5.

Upgrading Techniques for Clarifiers

There are two options for upgrading clarifiers:

1. modify the structural aspects of the clarifier
2. add chemicals

In considering physical or structural modifications to the clarifier, four possibilities exist:

1. optimize the aeration tank-clarifier relationship
2. modify the inlet-outlet configuration
3. control problem sludges
4. use of tube, lamella or other structural assists.

Figures 6,7 and 8 show various alternatives for inlets and outlets for circular clarifiers. Of the three alternatives, circular/centre feed, circular/rim feed - centre take off and circular/rim feed - rim take off, experience has shown that the latter gives optimum process performance.

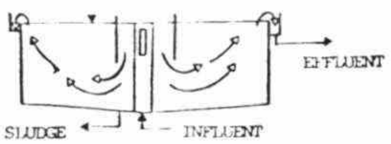


FIGURE 6 CIRCULAR CLARIFIER TYPES -- CENTRE FEED/RIM TAKE-OFF

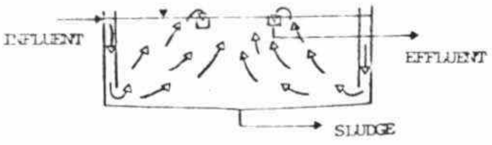


FIGURE 7 CIRCULAR CLARIFIER TYPES -- CIRCULAR RIM FEED/CENTRE TAKE-OFF

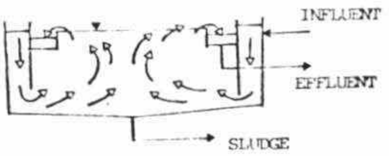


FIGURE 8 CIRCULAR CLARIFIER TYPES -- CIRCULAR RIM FEED/RIM TAKE-OFF

It should be noted that tube or lamella settlers will neither remove colloids nor promote flocculation. In every case the structural characteristics of the existing tanks will determine the feasibility of installing these devices. Frequently they are installed over a portion of the tank near the outlet. Some fouling of these devices due to biological growth has been experienced.

The upgrading of clarifiers using chemical techniques results in the following:

1. increased SS/BOD_5 removal;
2. reaction with alkalinity and soluble orthophosphates to form the respective metallic hydroxides or phosphates;
3. destabilization of colloids;
4. good flocculation/settling.

The question of when chemical addition to primary clarifiers as an upgrading technique should be practiced remains to be answered. Experience has shown that when any of the conditions as summarized below exist, then serious consideration of chemical upgrading should be given.

1. varying or intermittent flows are experienced;
2. limited available space for additional clarifiers;
3. industrial wastes are present which may interfere with biological treatment;
4. organic overloading;
5. interim improvements are required before new facilities are put on stream.

It is extremely important to recognize that by changing one unit operation, a cascading effect on the other downstream operations will occur.

For example, if the MLVSS are adjusted so that an F/M ratio of 0.2 results, then nitrification could occur and adequate resere aeration capacity must be available. Otherwise serious process difficulties will occur. Approximately 5 kg O_2 are required to oxidize 1 kg NH_4. Another effect of upgrading the primary clarifier for more efficient solids removal is that the organic load to the aeration basin decreases. This calls for more careful sludge management to ensure process stability.

Effluent Polishing

If overall process performance is to be increased by 10 to 20% so far as BOD BOD_5/SS removal is concerned, then consideration to such polishing processes as the following should be given:

1. polishing lagoons
2. filtration
3. microscreening
4. activated carbon

The use of activated carbon must have process effluent improvements other than BOD_5 and SS, otherwise if would not be cost competive with the other three alternatives identified above.

Upgrading Using Pre-Aeration & Post-Aeration Technology

These aeration technologies may be applied for the following reasons:

Pre-aeration:	- odour control - treatability improvement

The point of application is most frequently the grit chamber. Some flocculation and grease separation have been observed to be desirable side benefits. Frequently, chemicals are also added to the grit chambers which result in enhanced BOD_5/SS/P removal in the primary process.

Post-Aeration:	- to provide additional DO to the receiving stream (increase effluent DO from 2.0 to 4.0 mgL^{-1}).

Four different methods of post-aeration have been practiced in the past:

1. diffused
2. turbine
3. cascade
4. U-tube

There is no clear picture though with respect to the relative advantages/disadvantages and costs of these alternatives. Each situation needs to be evaluated on its own merit.

Upgrading of Sludge Handling

Process upgrading opportunities not only exist in the area of constituent removal technology - but also in sludge handling. Examples are:

1. reducing sludge volumes prior to digestion;
2. combined thickening and equalization prior to sludge dewatering;
3. reducing the sludge volume for handling.

Gravity thickening should always be considered when faced with a sludge handling upgrading problem (opportunity). Experience has shown that a gravity thickener feed solids concentration of 0.5 to 1.0% results in optimum process response. For example, at one modified aeration activated sludge plant, the combined primary and waste activated sludge had a feed concentration of less than 1%, the thickener underflow increased to approximately 11%. At another plant primary and conventional waste activated sludge were successfully thickened to 6%.

Figure 9 illustrates quite dramatically the impact of sludge solids concentration on the required digester volume. By increasing the solids feed to the digester from 3.5% to 7.5% the digester volume can be reduced by 50%.

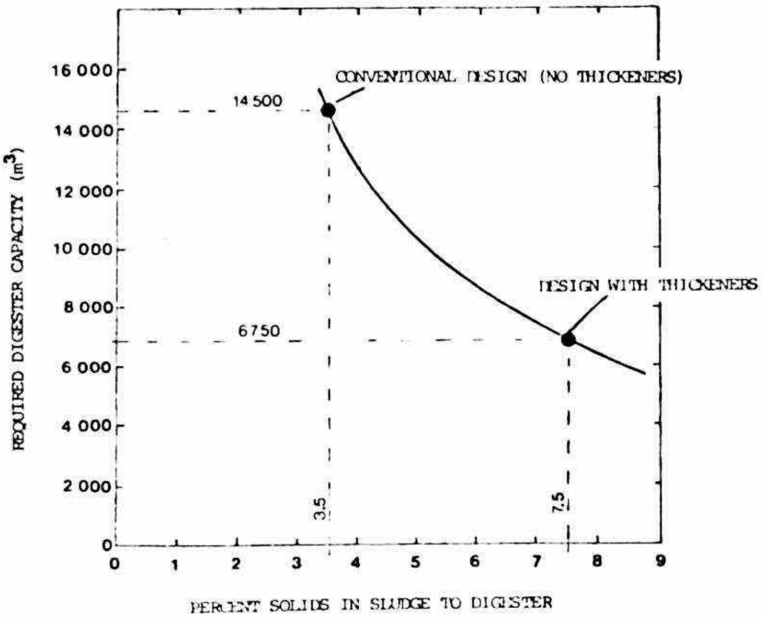


FIGURE 9 EFFECT OF SLUDGE THICKENING ON DIGESTER VOLUME

Centrifugal thickening has also been used for upgrading existing sludge handling facilities. It is another process alternative which should be considered. It is flexible, can be used to upgrade an overloaded anaerobic digester by reducing the feed sludge volume and thus increase the digester detention time or alternatively, handle an increased volume of sludge resulting from other plant capacity expansions. It is critical however, to ensure that adequate digester mixing capacity is available. Centrifugal thickening could also be used to supplement existing overloaded gravity thickeners. An example of centrifugal thickening and its impact on upgrading the digester capacity is shown in Table 5. The net result is that the digester HRT has been increased by 100%.

TABLE 5 IMPACT OF CENTRIFUGAL THICKENING ON DIGESTER HRT

DESCRIPTION		OVERLOADED	UPGRADED	
PRIMARY SOLIDS	kg	7000	7000	
PRIMARY SLUDGE VOL.	m^3d^{-1}	-	140	(6%)
WASTE ACTIVATED SOLIDS	kg	4400	4400	
WASTE ACTIVATED VOL.	m^3d^{-1}	-	105	(5%)
COMBINED SOLIDS	kg	11400	11400	
COMBINED SLUDGE VOL.	m^3d^{-1}	455	245	(5.6%)
DIGESTER HRT	days	11.25	21	

Waste Activated Sludge Recycled to Primary Clarifier (before upgrading)

In upgrading existing anaerobic digestion facilities from a low-rate to a high-rate process, the following conditions must be met:

1. thicken solids to increase volatile solids load and HRT
2. ensure that digester contents are completely mixed
3. attempt to feed and withdraw solids uniformly
4. ability to control the temperature between 30-35°C
5. the initial HRT should be at least 15 days.

Existing sludge dewatering equipment such as vacuum filters for example, can be upgraded by making changes to the following:

1. feed solids concentration
2. filter medium
3. sludge conditioning

Centrifugal dewatering can also be used in upgrading sludge handling facilities. When persuing this alternative, the efficiency of existing grit removal and screening facilities must be ensured. Abrasion due to the presence of grit can be reduced by putting cyclonic grit removal ahead of the centrifuge. Inadequate removal of screening will result in the plugging of the centrifuge inlet.

SUMMARY

There is little doubt that treatment plant upgrading through process modifications is a cost-effective method of providing increased capacities at existing plants. Upgrading technologies will play an increasingly important role in the future as energy and money become more expensive.

A thorough understanding of the principles involved in the various unit operations and processes and recognition of their interdependance, when coupled with an in-depth analysis of existing plant data will permit the process engineer to quickly identify opportunities for plant upgrading at minimum cost.

URBAN PHOSPHORUS MANAGEMENT

THE STRATFORD EXPERIENCE

BY

G. ZUKOVS & F. ENGLER
WASTEWATER TREATMENT SECTION
POLLUTION CONTROL BRANCH
ONTARIO MINISTRY OF THE ENVIRONMENT

Summary

The Stratford-Avon River Environmental Management Project (SAREMP) was initiated to resolve water quality problems in the Avon River. Particular emphasis has been placed on control of phosphorus from both urban and rural sources.

The present study is a component of the SAREMP urban investigations intended to prescribe the most cost-effective combination of urban control measures for the management of phosphorus.

Two aspects of the work are presented:

* the characterization of phosphorus loadings from both urban dry and wet weather sources.
* the feasibility and cost-effectiveness of achieving very low (0.1 mg/L total phosphorus) phosphorus concentrations in the WPCP effluent.

Four major sources of phosphorus loadings were quantified. They were as follows:

* WPCP effluent.
* WPCP wet weather bypass.
* stormwater runoff.
* dry weather discharges from storm sewers (consisting chiefly of industrial cooling water and groundwater seepage).

Results of this assessment indicated that 41% of the seasonal total phosphorus loading originated from the WPCP effluent with the remainder reasonably equally distributed among the other sources.

A WPCP phosphorus treatability study was accordingly initiated. The study evaluated by jar tests and subsequently through fullscale trials the minimum effluent phosphorus concentration reasonably achievable. A two stage removal process was tested. In the first process stage approximately 68 mg/L alum was applied to the aeration effluent with an additional 15 mg/L alum applied in advance of the effluent filter. An average effluent total phosphorus of 0.1 mg/L was consistently achieved using this process scheme.

As a consequence of the second stage alum addition, filter backwash requirements nearly doubled with a 2.3 times increase in solids loading returned to the primary clarifiers. Operating cost increases due to dual stage operation were estimated to be approximately $1,800 per month. Capital costs to provide a second chemical dosing system were estimated to be modest.

1.0 Introduction

In April, 1980 the two year Stratford-Avon River Environmental Management Project (SAREMP) was initiated. This project represented a broadly based river basin management study aimed at resolving a variety of water quality problems in the Avon River. These problems had been identified in the earlier Thames River Basin Management Study (1975) (1) and a subsequent study entitled Impact of Waste Inputs on the Quality of the Avon River (1979) (2) as originating from both urban and rural sources. The present paper discusses one component of work carried out under the SAREMP and details some aspects of a study of the management of urban phosphorus sources in the City of Stratford.

2.0 Description Of Study Area

Stratford is a community of about 27,000 people, situated on the Avon River in southwestern Ontario. Stratford has an urban area of approximately 2,000 ha which is divided into 700 ha (north-shore), and 1,300 ha (south-shore) areas by the river. Land use is primarily residential/commercial with roughly one third given to light industry, mostly metal fabricating and finishing.

The problems identified in the Avon River occur downstream of the urban area. Heavy summertime growth of aquatic plant biomass consisting principally of the Alga Cladophora, moss and pondweed has been observed. Combined with summertime low stream flows this condition has lead to significant swings in diurnal dissolved oxygen levels in the river.

The observations are characteristic of excessive nutrient inputs to the river, principally the input of bio-available phosphorus. Accordingly, one of the chief activities of the Stratford-Avon project was to identify urban and rural phosphorus sources and to formulate a comprehensive strategy for their management.

3.0 Urban Phosphorus Management Study Plan

The study plan adopted for the development of an urban phosphorus control strategy is shown in Figure 1. The plan consists of five key elements as follows:

1. Source characterization of urban phosphorus loadings.
2. Phosphorus treatability studies at the WPCP.
3. Formulation of an urban phosphorus control strategy.
4. Integration of this strategy with the rural phosphorus management program.
5. Formulation of an overall phosphorus management plan for the Avon river basin.

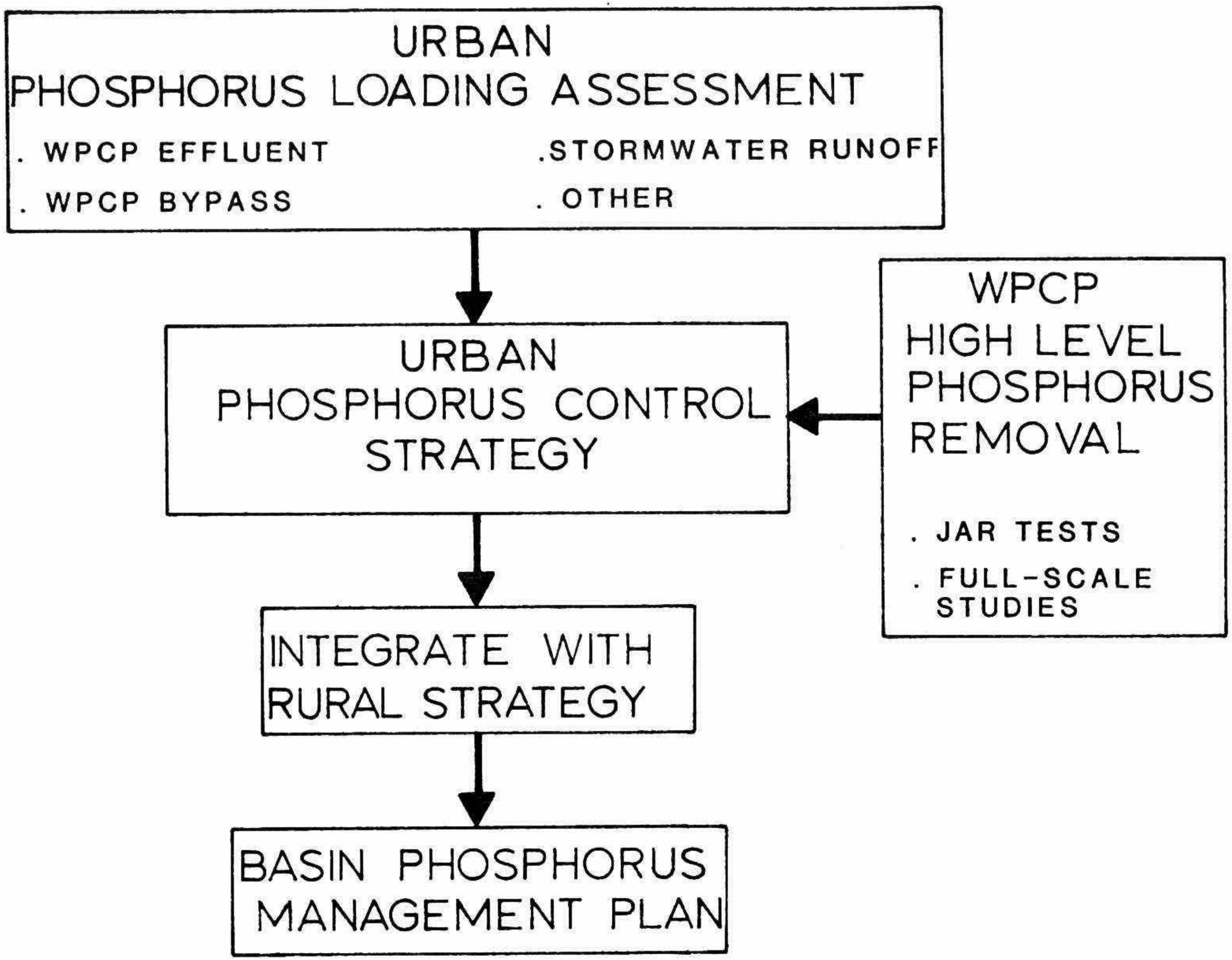

URBAN PHOSPHORUS LOADING ASSESSMENT
. WPCP EFFLUENT
. WPCP BYPASS
.STORMWATER RUNOFF
. OTHER
URBAN PHOSPHORUS CONTROL STRATEGY
WPCP HIGH LEVEL PHOSPHORUS REMOVAL
. JAR TESTS
. FULL-SCALE STUDIES
INTEGRATE WITH RURAL STRATEGY
BASIN PHOSPHORUS MANAGEMENT PLAN

FIGURE 1:

URBAN PHOSPHORUS MANAGEMENT STUDY PLAN

The balance of this paper deals with the first two key elements, namely loading assessment and treatability studies.

4.0 Urban Phosphorus Loading Assessment

Field monitoring and sampling was conducted during the summer periods (May to September) of 1980-1981 to determine dry weather loadings. Wet weather loadings for the same periods were computed using simulation modelling (STORM model).

The nature of the dry and wet weather sources of phosphorus originating from the urban area has been depicted schematically in Figure 2.

In dry weather three sources have been identified:

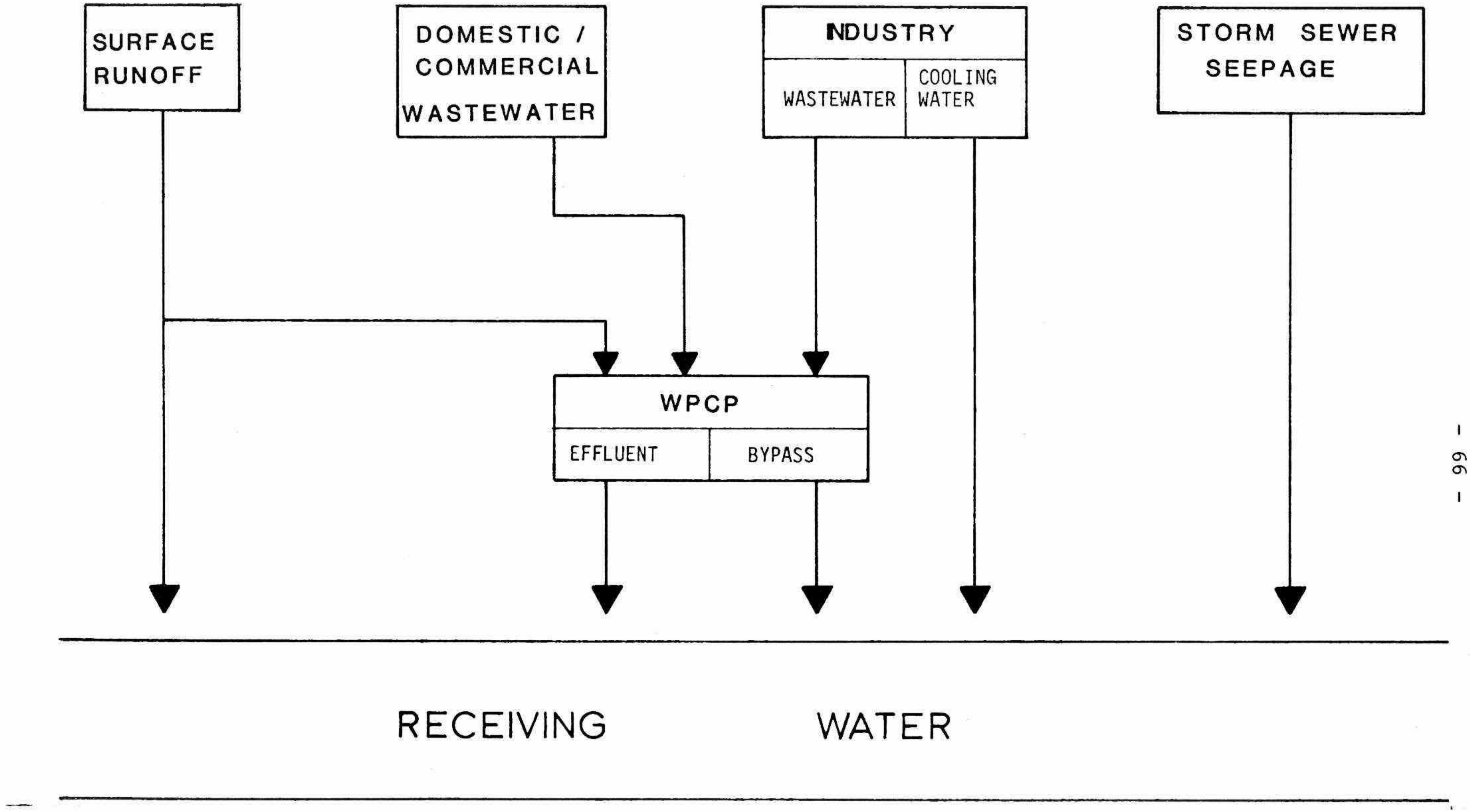
1. The WPCP effluent which will be discussed in greater detail further on.
2. Groundwater seepage from the storm drainage system.
3. Industrial cooling water.

(Note - for purposes of subsequent discussion sources 2 and 3 have been lumped together into a category termed other sources).

In wet weather two sources have been identified:

1. Surface runoff discharged through the storm sewer outfalls.
2. WPCP bypass flows consisting of both raw sewage and primary effluent. Bypassing is the result of substantial inflow and infiltration into the sanitary sewer system.

FIGURE 2:
URBAN POLLUTANT SOURCES



4.1 Urban Seasonal Average Total Phosphorus Loadings

The distribution of seasonal total phosphorus loadings based on results of the 1980-1981 surveys has been calculated and is indicated in pie chart form in Figure 3. The following comments should be made regarding the derivation of the distribution and the attached significance.

1. Only the summer season of May to September has been studied since this is the known critical period of plant growth (available daylight, temperature, etc).
2. The basis of the assessment is predicated on total seasonal loadings. The rationale underlying this approach stems from the belief that the stream bio-mass is able to take up and store phosphorus for later use.
3. Using the aforementioned approach it can be seen that the WPCP effluent is the dominant loading with the sum of WPCP bypass plus storm runoff contributing an amount roughly equalling the plant loading.

The magnitudes of the loadings from which the pie chart has been drawn up are summarized in Table 1. Seasonal loadings of total and soluble phosphorus are indicated. Average concentrations of the various sources and the equivalent concentration of the sum of the sources is also presented.

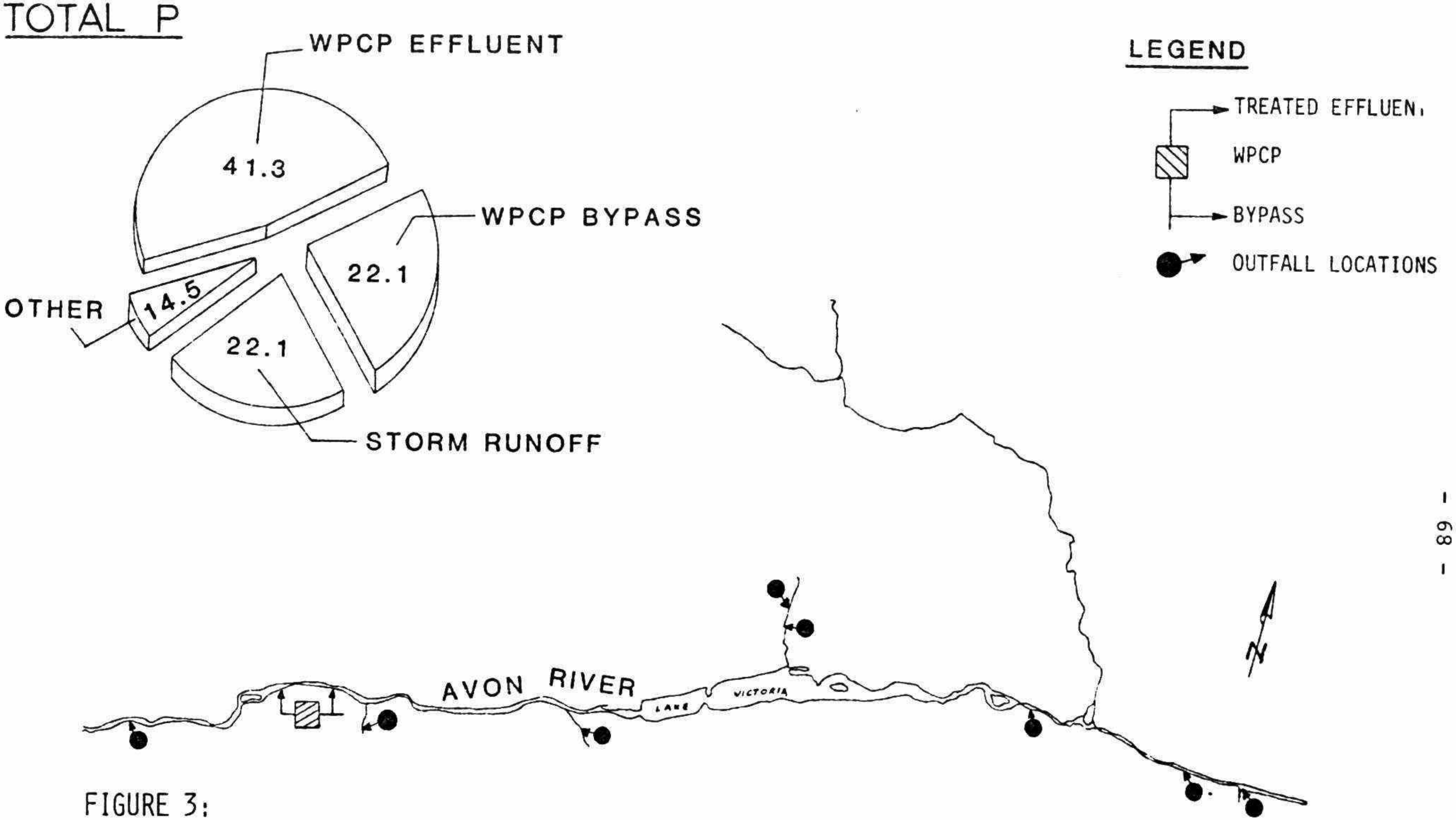


FIGURE 3:

URBAN SEASONAL AVG. LOADINGS MAY - SEP. 1980 - 1981

Table 1

Urban Seasonal Average Loadings

May-September 1980-1981

	WPCP Effluent	WPCP Bypass	Storm Runoff	Other	Total
Volume ($10^3 m^3$)	2,600	100	1,000	1,400	5,100
Total P, Kg	900	500	500	300	2,200
Avg Conc.*	0.35	5.0	0.50	0.21	0.43
Soluble P, Kg	300	200	200	170	870
Avg Conc.*	0.12	2.0	0.20	0.12	0.17

* Units : mg/L

As can be seen from Table 1, a high degree of phosphorus removal is regularly achieved at the WPCP during the summer. The average effluent total P of 0.35 mg/L, represents approximately 14 kg/day of phosphorus. It should be noted that the dry weather portion of the flows represents about 67% of the average dry weather river flows. On the basis of this analysis of loadings it can be concluded that the single largest and most controllable source of phosphorus is the WPCP effluent. Accordingly, treatability studies were conducted at the WPCP to determine the maximum feasible degree of phosphorus removal at that facility. Other work, not reported in this writing, has looked into the control of other phosphorus sources. This was done to achieve the most cost-effective overall control strategy for the urban sources.

5.0 WPCP High Level Phosphorus Removal Study

The study was carried out in three phases of activity:

1. A review was made of recent historical plant effluent data and operational records. Using this data and some recent literature findings a process scheme using two stage chemical addition (alum) for phosphorus removal was adopted.
2. A jar test study was used to determine the optimum second stage chemical dosage.
3. A fullscale trial was carried out to determine maximum phosphorus removal feasible and to assess operating costs and process impacts.

5.1 Historical Process Review

5.1.1 WPCP Process Schematic

The Stratford WPCP is a conventional activated sludge plant designed for a flow of 27,300 m^3/d. Figure 4 depicts a simplified process schematic for the facility.

The plant is provided with a tertiary effluent filtration stage which has been in operation since 1976. Four filter beds of anthracite and sand are available to handle ultimate design flows. Currently, only two beds are generally in service at one time.

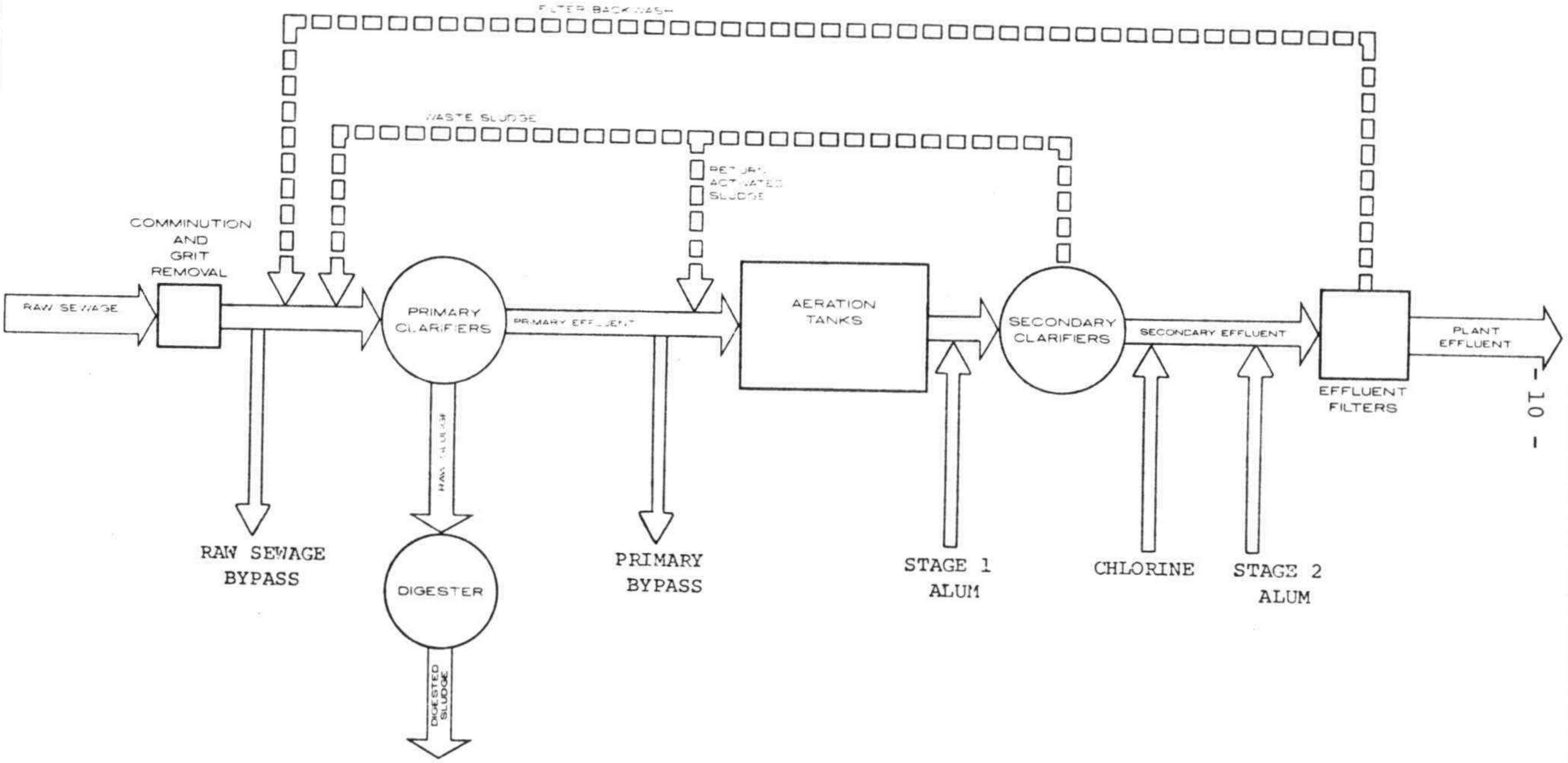


FIGURE 4:
CITY OF STRATFORD
WATER POLLUTION CONTROL PLANT

Alum is used as the phosphorus removal coagulant. Point of application is to the tail end of the aeration section. In the two stage process a second point of alum addition was made directly to the influent of the filter.

5.1.2 Typical WPCP Process Parameters

Several of the plant process parameters for 1980-1981 have been abstracted from the operating log and are summarized in Table 3. This table provides some background to typical process parameters for the Stratford WPCP.

Table 3

Typical Process Parameters

Stratford WPCP, 1980-1981

Biological

	SRT (Days)	F/M Ratio	SV1	30 Minute Settling
Summer	5.8	0.19	61	15
Winter	6.2	0.20	61	13

	Hydraulic Retention HRT (hrs)	Peak/Avg DWF	SSR[2] m/d	P Removal[1] Alum Dosage (mg/L)	Avg Al/P Ratio
Summer	6.4	1.4	21	47	1.0
Winter	7.3	1.2	24	25	0.6

Notes

1) As $Al_2(SO_4)_3\ 14\ H_2O$.

2) Secondary clarifier surface settling rate at average DWF.

As can be seen from the biological parameters the plant is operated in the conventional activated sludge range. The dry-weather final clarifier surface settling rates (SSR) are well within the Ministry guideline limits of 35 m/d.

Phosphorus removal practice varies winter to summer. Winter target level for phosphorus is 1.0 mg/L, whereas the summer target is 0.3 mg/L. It is interesting to note that this objective has been reached with average Al/P ratios significantly below typical Ontario values of 1.3. Perhaps one reason for the difference is the fact that at Stratford the ratios were based on primary effluent phosphorus loading rather than the more conventional raw sewage phosphorus loading.

5.1.3 Average 1980-1981 Tertiary Effluent Quality

Table 4 is a summary of average tertiary effluent quality for 1980-1981 for the Stratford WPCP.

Table 4

Average Effluent Quality

Stratford WPCP 1980-1981

	BOD	SS	TOTAL P	SOLUBLE P	NH_3-N
Summer	5.3	4.6	0.35	0.16	4.3
Winter	5.7	3.6	0.72	0.50	5.4

(Units: mg/L).

It is evident that a very high dry weather effluent quality is achieved at the plant. During the summer the plant partially nitrifies, although the summer ammonia values represent the average value for the whole May to September period. Mid-summer values in July and August typically are as low as 1.0 mg/L. The summer average total phosphorus slightly exceeds the target value of 0.30 mg/L but the winter value is significantly better than the target value of 1.0 mg/L.

To show the degree of variability of the effluent phosphorus levels the distribution of the analytical results has been plotted on a normal probability scale, shown in Figure 5. Here the obvious effect of filtration on phosphorus removal can be seen. Comparing the secondary and tertiary effluent phosphorus residuals indicates a 0.13 mg/L difference for winter data and a 0.19 mg/L difference for summer data at the 50% frequency. The summer data also indicate a more consistent effluent quality. This is due to the higher alum dosages used from May to September.

The metal to phosphorus (Al/P) ratios for the 1980-1981 summer and winter periods were calculated and are depicted in Figure 6. Examination of this figure shows that an effluent total phosphorus concentration of 0.3 mg/L could readily be reached, but below this value the curves tended to become asymptotic. As a result, the addition of more alum in a single stage would likely be inefficient, producing little improvement in effluent phosphorus per extra unit of dosage.

FIGURE 5:
EFFLUENT TOTAL P
STRATFORD WPCP 1980-1981

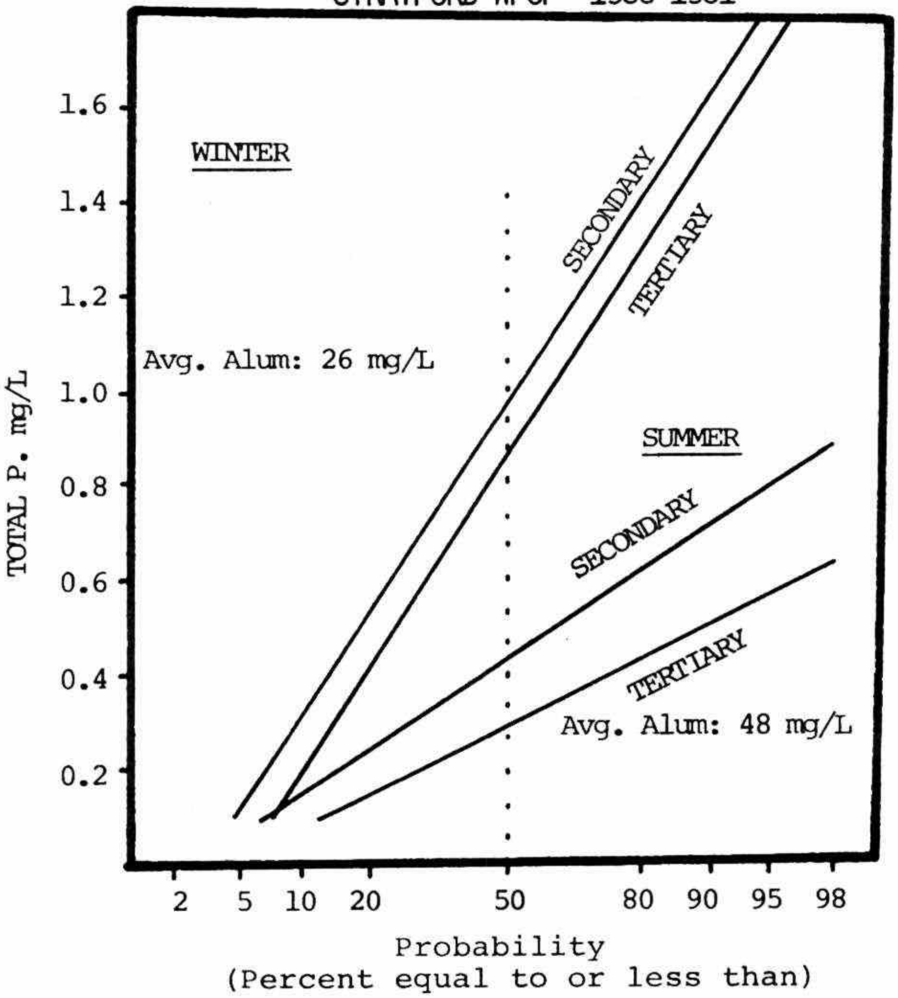
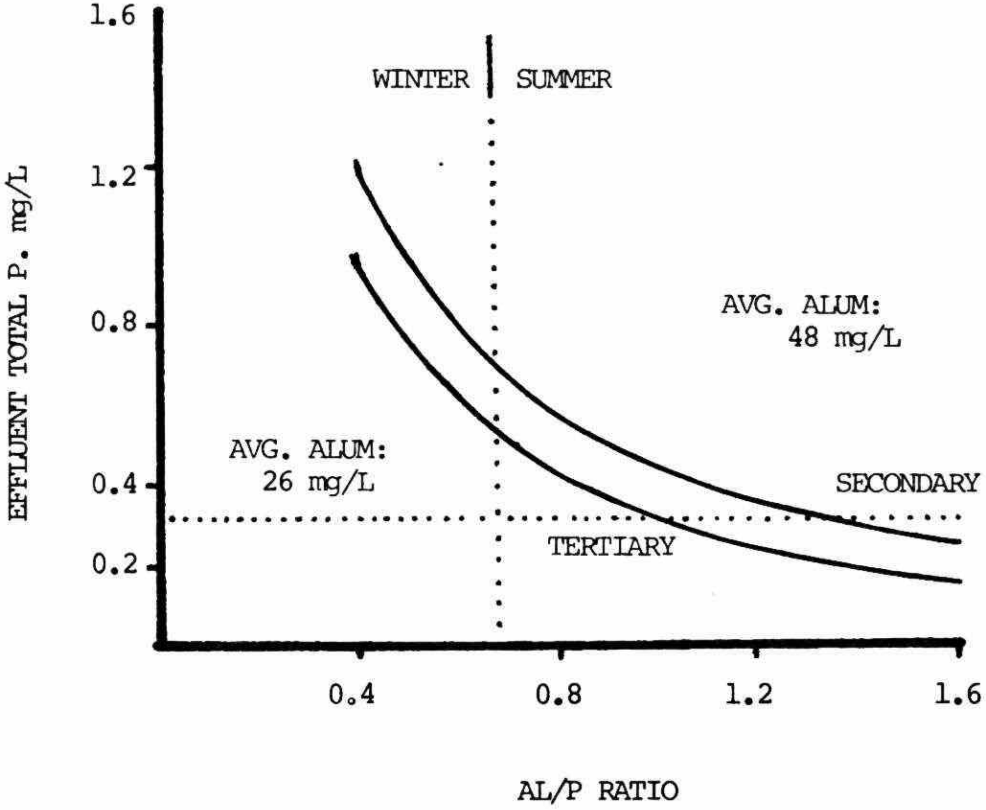


FIGURE 6:

AL/P RATIOS VS. EFFLUENT TOTAL P.

STRATFORD WPCP 1980-1981



5.2 Dual Stage Chemical Addition

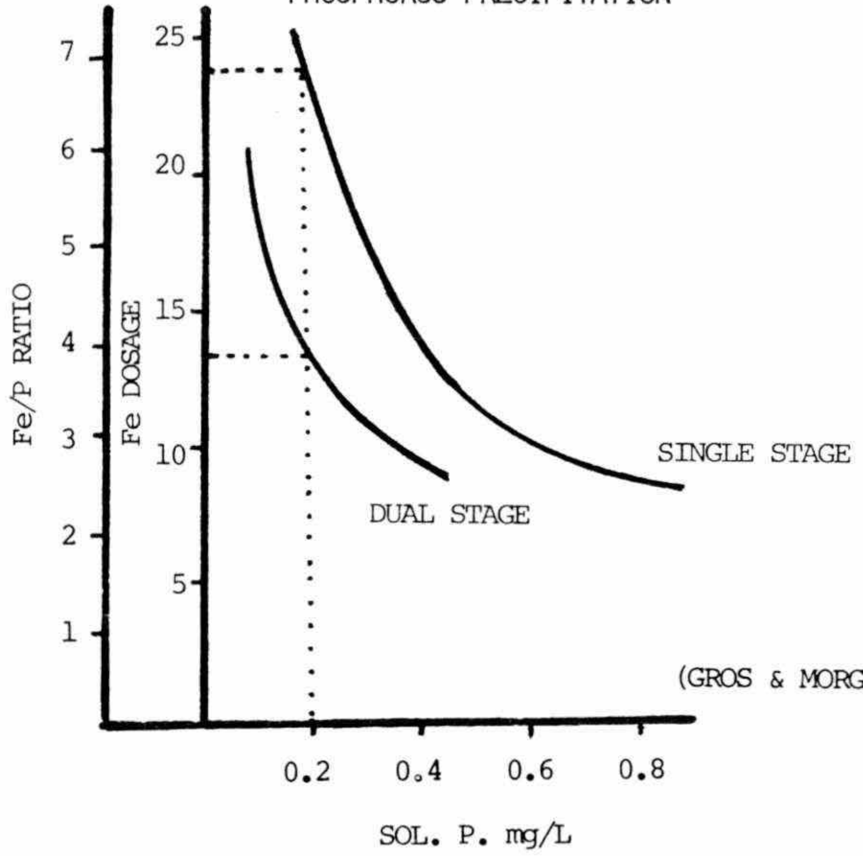
A literature search of recent high level phosphorus removal experiments brought to light recent work by Gros & Mörgeli(3), done in Switzerland in 1980. In their experiments the authors employed a dual stage chemical addition scheme, using ferric chloride, to reduce phosphorus levels economically to a lower practical limit in an activated sludge/effluent filtration facility. In their scheme, Gros & Mörgeli added coagulant at low dosage directly to the filter inflow in addition to coagulant applied at conventional locations in the treatment plant.

The effect of such dual stage chemical addition is shown in Figure 7, which has been abstracted from the original paper. It can be seen that a target level of 0.2 mg/L soluble P in the effluent required approximately 14 mg/L iron for the two stage process versus 24 mg/L for the single stage process.

For the work at Stratford, due to time limitations, only one of many possible dual stage addition combinations was tried. The existing chemical dosage to the aeration system (about 50 mg/L alum) was retained and the optimum second stage alum dosage to the filter identified through a modified jar test procedure.

FIGURE 7:

COMPARISON OF SINGLE AND DUAL STAGE PHOSPHORUS PRECIPITATION



(GROS & MORGELI, 1980)

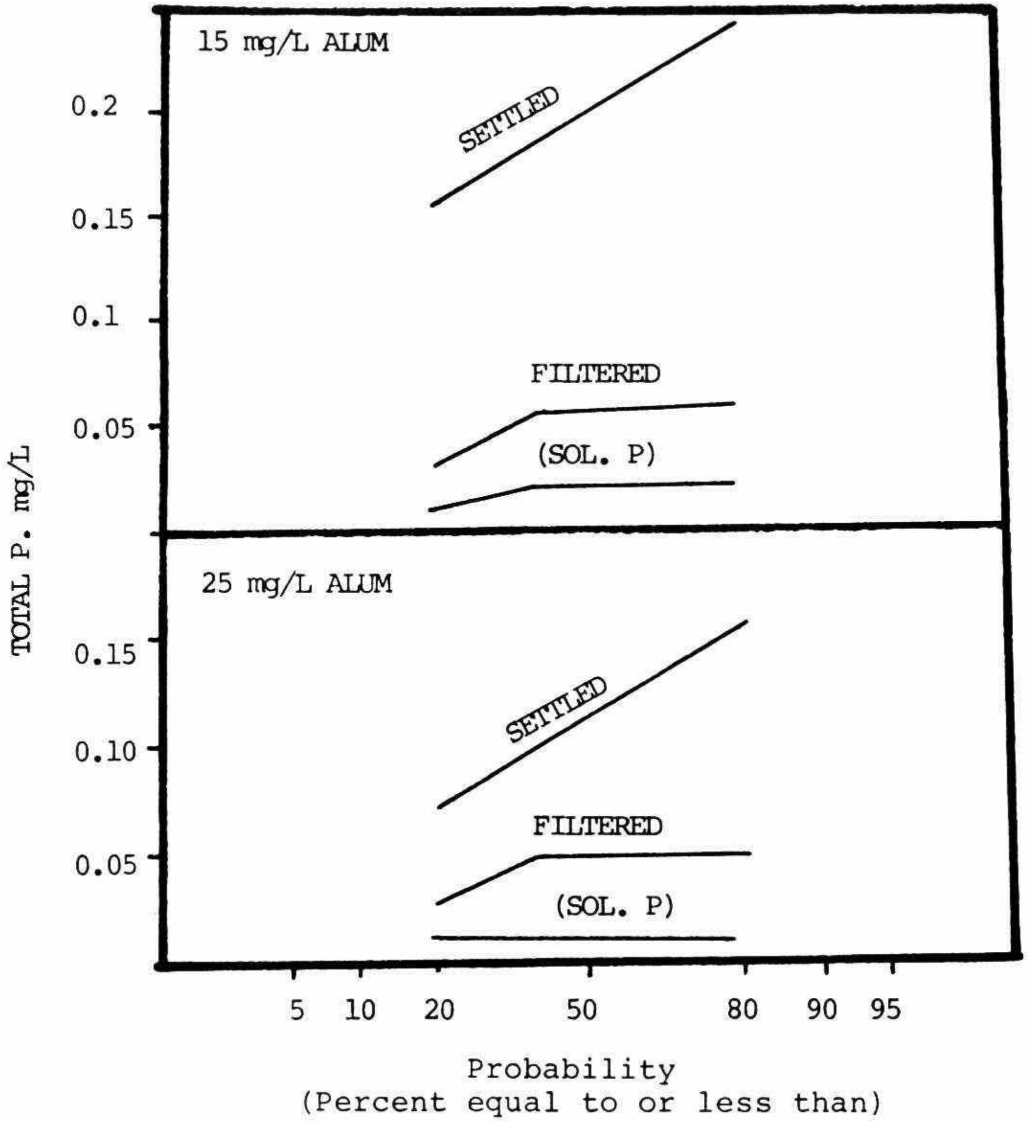
5.3 Jar Testing

Jar testing, using the conventional wastewater treatment procedure, was carried out applying a range of alum dosages to the already alum treated secondary effluent. The jar test dosages evaluated ranged from 5 mg/L to 25 mg/L alum. Analyses of soluble and total phosphorus were done on jar decantate and filtrate after 30 minutes of settling.

Figure 8 illustrates the findings of jar testing on a probability plot. The three lines represent the three phosphorus species present, namely particulate, condensed (or non-reactive soluble), and soluble ortho phosphorus. From the top portion of Figure 8 it can be seen that for a 15 mg/L alum dosage 1) very little soluble phosphorus remains, and 2) the actual total phosphorus in fullscale operation can be expected to be intermediate between "settled" and "filtered". Increasing the applied alum dosage to 25 mg/L yields no improvement for the filtered and soluble phosphorus and only marginal improvement for the particulate or "settled" phosphorus.

Based on jar test results and because of some concerns with respect to the effluent filter operation during unattended night-time hours it was decided to employ the 15 mg/L alum dosage at the second point of addition.

FIGURE 8:
JAR TEST RESULTS STRATFORD WPCP JUNE 1981



5.4 Fullscale Testing

The screw pump feeding the filter was chosen as the second point of alum addition. Pump action and the subsequent drop shaft at the head end of the filter bed inlet channel provided good mixing and some flocculation. Flocculation continued in the effluent ponded on the filter media.

Both chemical feed pumps were manually adjusted to plant flow. Dosage control to the aeration tank feed point was somewhat poor due to mechanical problems with the feed pump. On average 68 mg/L alum were actually added; combined with the 15 mg/L to the filter total plant alum dosage was 83 mg/L.

Samples were taken by automatic 24 hour composite sampler over the test period. Table 5 summarizes the quality of various treatment plant streams during the dual stage trial period from July 14-28.

Table 5

Analytical Results

Dual Stage Phosphorus Removal

Stratford WPCP, July, 1981

	BOD	SS	TOTAL P	SOLUBLE P	NH_3-N
Raw Sewage	116	227	5.5	3.3	18
Primary Effluent	77	96	4.5	2.2	18
Secondary Effluent	5	11	0.42	0.12	1.7
Tertiary Effluent	5	5	0.10	0.03	0.3

Units: mg/L

It is noteworthy that a 0.1 mg/L total phosphorus tertiary effluent was achieved with a simultaneous very low soluble phosphorus concentration. Furthermore, it is interesting that even at the higher than usual alum dosage to the aeration section the secondary effluent total phosphorus remained above the summertime target level of 0.3 mg/L. The same observation was already made during review of the 1980-1981 historical process parameters.

Examining the phosphorus data for effluent phosphorus on a probability plot as shown in Figure 9 makes it clear that

1. The soluble phosphorus in the effluent has been reduced to practical limits.
2. The total phosphorus in the filtered effluent exhibits less variability than with sedimentation alone.

5.5 Filter Operation

During the period of testing, filter operation was monitored closely to determine the impact of dual stage phosphorus removal upon the filtration stage. Operating staff were requested to maintain the usual two filters on/two filters off mode of operation if possible. However, staff became concerned that insufficient safety margin against filter overflow was maintained over the unattended nighttime period, and typically at 8:00 PM a third filter was brought into service. By 10:00 AM of the following morning the normal two filter mode of operation had resumed.

FIGURE 9:
DUAL STAGE PHOSPHORUS REMOVAL RESULTS
STRATFORD - JULY 1981

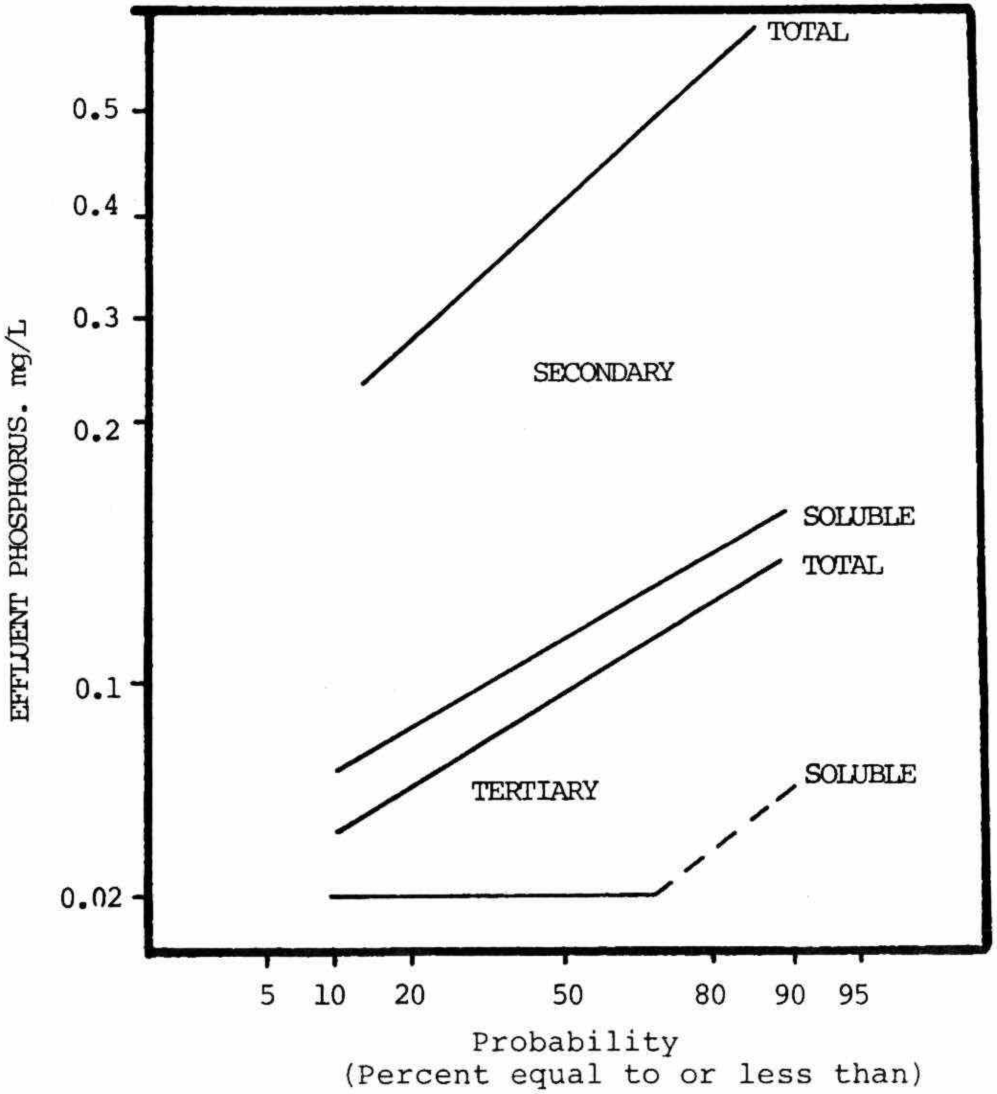


Table 6 summarizes filter loading data during the dual stage phosphorus testing. Both solids and volumetric loadings are well within MOE guidelines values in all cases (MOE Guidelines - solids 7.2 kg/m^2.d; volumetric - 12 m/h). There was, however, some doubt as to the necessity of employing the additional filter during the nighttime period. A longer evaluation would be required to firmly establish the mode of filter operation. Clearly it is evident that the presence of alum on the filters has increased the solids loading, necessitating 85% more frequent backwashing. However, only a longer term operational evaluation will resolve this issue.

The solids returned to the primaries show a dramatic 260% increase for the dual stage process. However, this loading still represents only 10% of the total load to the digesters. The impact on digester operation should be determined from observations over a longer term test period. In neither of the above cases, are the impacts sufficient to be of particular concern; although increases in operating costs would result. Adding the cost of additional coagulant to the costs resulting from increased filter operation gives an overall estimated increase in monthly costs of $1,800. Over the May to September season a cost of $9,000 per annum would be incurred. Capital costs for implementation of dual-stage removal would be fairly modest, comprised of new feed equipment and possibly an additional storage tank.

Table 6

Filter Hydraulic And Solids Loadings

Observed During The July, 1981 Test Period

FILTERS IN SERVICE	HYDRAULIC LOADING	SUSPENDED SOLIDS LOADING
2	10.4 m/h	6.5 kg/m^2.d
3	7.0 m/h	4.3 kg/m^2.d

Other operating parameters have been calculated and are compared with single stage alum addition filter performance as determined from the plant operating log.

Table 7

Comparison Of Filter Performance

For Single And Dual Stage Phosphorus

Removal, Stratford, July, 1981

	BACKWASHES PER DAY	DAILY BACKWASH VOLUME (m^3)	RUN TIME FILTER hrs/d	SOLIDS RETURNED TO PRIMARIES (kg/d)
Single Stage	2	600	48	95
Dual Stage	3.7	1,100	61	350
% Increase	85	85	27	260

6.0 Conclusions

1. An effluent concentration of 0.1 mg/L total phosphorus is achieveable at the Stratford WPCP by the application of alum in a dual-stage process. This process uses in the first stage 68 mg/L alum applied to the aeration effluent followed by a second stage of 15 mg/L alum applied directly in advance of the effluent filter.
2. Filter operation is impacted by the application of alum in the second stage. Filter solids loading and backwash frequency and volume are increased. Filter operation is, nevertheless still well within accepted guidelines.
3. The increase in operating costs due to the use of the dual-stage process are estimated to be \$1,800 per month or \$9,000 per season (May to September). Capital costs of implementing the dual-stage process are felt to be modest.

References

1. Ontario Ministries of Environment and Natural Resources, 1975, Water Management Study, Thames River Basin.
2. Ontario Ministry of the Environment, 1979, Impact of Waste Inputs on the Quality of the Avon River.
3. Gros, H., and Mörgeli, B., "Optimal Advanced Treatment and Phosphorus Removal By Deep Bed Filtration", project Water Technology, Volume 12, pp. 315-332, Pergamon Press, 1980.

ROLE OF TREATABILITY STUDIES IN THE UPGRADING OF A MUNICIPAL/INDUSTRIAL TREATMENT PLANT

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For presentation at

SEMINAR FOR
WASTEWATER TREATMENT PLANT UPGRADING

Sponsored by

THE POLLUTION CONTROL ASSOCIATION OF ONTARIO
and
THE ONTARIO MINISTRY OF THE ENVIRONMENT

March 10, 1982

1 INTRODUCTION

The Township of Woolwich, Elmira Water Pollution Control Plant (WPCP) is scheduled to be expanded from its present design capacity of 0.68 MGD to 1.0 MGD. In addition to the requirement for expansion of hydraulic capacity, treatment upgrading is required in three areas:

1) to meet existing MOE effluent objectives,
2) to meet new MOE effluent objectives,
3) to meet new MOE sludge disposal guidelines.

Because of the relatively large proportion of flow (up to 0.18 MGD), and organic loading (67% of the average design BOD_5 loading) contributed to the Elmira WPCP by the Uniroyal Chemical Ltd. plant located in Elmira and because of the presence of high concentrations of ammonia, total Kjeldahl nitrogen, phenolics and other trace organic components in the Uniroyal wastewater, it was recommended by the Ministry of the Environment that treatability studies be conducted to develop design parameters for the treatment plant expansion/upgrading. The specific design areas requiring treatability studies at the Elmira WPCP were:

1) Nitrification
2) Anaerobic Digestion
3) Sludge Dewatering

The objectives, methodology and results of each of these treatability studies are presented in the following sections of this paper.

2 NITRIFICATION TREATABILITY STUDY

2.1 Objectives

In June 1979, Canviro Consultants Ltd. was authorized by the Regional Municipality of Waterloo to proceed with a treatability program to develop process design criteria for expanding the capacity and upgrading the efficiency of the Elmira WPCP plant to meet the Ministry of the Environment's effluent quality objectives summarized in Table 1.

Specific objectives of the treatability study included:

1. Evaluate solids retention time (SRT) as it affects nitrification and relates to return sludge capacity and secondary clarifier surface area.

2. Determine the required hydraulic retention time (HRT) for treatment. (i.e., aeration basin volume requirments).
3. Determine biological process stability over the anticipated range of Town to Uniroyal flow combinations.
4. Develop operational guidelines for the plant. (MLSS, aeration DO, sludge wasting schedule, etc.).
5. Compare the treatability of the Town wastewater alone, to practical combinations of Town to Uniroyal flows to establish a base for cost sharing calculations.

TABLE 1. MOE EFFLUENT QUALITY OBJECTIVES FOR ELMIRA, WPCP

PARAMETER	OBJECTIVES
BOD_5	7.5 mg/L (75 LB/DAY)
SS	15.0 mg/L (150 LB/DAY)
PHOSPHORUS	1.0 mg/L (10 LB/DAY)
PHENOLS	6.5 ug/L (0.065 LB/DAY)
pH	6.5 - 8.5
TKN (Apr 1 - Oct 31)	3.5 mg/L (35 LB/DAY)
NH_3 (Nov 1 - Mar 31)	7.5 mg/L (75 LB/DAY)

The treatability program was divided into several test runs with major emphasis being placed on meeting the nitrification requirements. However, achievement of the BOD_5 and phenolics objectives was also a major concern of the study.

2.2 Methodology

The overall experimental design of the study is illustrated in Table 2. The process parameters evaluated in the treatability program included:

(a) Temperature (T) - summer (20°C) and winter (5-8°C).
(b) Solids Retention Time (SRT).
(c) Hydraulic Retention Time (HRT).
(d) Town to Uniroyal wastewater combination ratios (i.e., Town only and 5:1- and 2:1 which represented the average design and "worst case" ratios.

A temperature controlled system with three parallel treatment units each consisting of a premix tank, aeration basin, clarifier, and effluent storage tank, with continuous feed to the aeration basins and continuous sludge return was

TABLE 2

EXPERIMENTAL PROGRAM FOR NITRIFICATION TREATABILITY STUDY AT THE ELMIRA WPCP

PERIOD	NO. OF WEEKS	DESCRIPTION AND NOMINAL CONDITIONS
Sept. 24 - Nov. 11, 1979	7	Acclimatization to Run 1 conditions HRT SRT TEMP. A 2:1 24h 30d 20°C B 5:1 24h 30d 20°C C Town 12h 10d 20°C
Nov. 12 - Dec. 11, 1979	5	Run 1 - Steady State
Dec. 12 - Jan. 20, 1980	5	Acclimatization to Run 2 conditions HRT SRT TEMP. A 2:1 24h 50d 6°C B 5:1 24h 50d 6°C C Town 12h 25d 6°C
Jan. 21 - Mar. 13, 1980	8	Run 2 - Steady State
Mar. 14 - Apr. 9, 1980	4	Acclimatization to Run 3 conditions HRT SRT TEMP. A 2:1 12h 50d 6°C B 5:1 12h 50d 6°C C Town 6h 25d 6°C
Apr. 10 - May 8, 1980	4	Run 3 - Steady State
May 9 - May 23, 1980	2	Run 4 - Non Steady State HRT SRT TEMP A 2:1 18h 50d 6°C B 5:1 8h 50d 6°C C Town 4h 25d 6°C
May 24 - June 12, 1980	3	Run 5 - Non Steady State HRT SRT** TEMP. A 2:1 18h* 25d 6°C B 5:1 12h 40d 6°C C Town 6h 20d 6°C
June 13 - July 31, 1980	6	Run 6 - Variable Nitrogen, BOD and Phenolics Loading HRT SRT TEMP A 2:1 36h 25d Ambie B 5:1 18h 60d 6°C C 5:1 18h 30d 6°C

* Changed to 24 hr. on June 4, 1980.

** SRT was reduced to the indicated values at which time process failure was imminent.

custom designed for this study. Equipment fabrication and site preparation were completed in August and CANVIRO'S mobile laboratory was moved onto the site, beside the Uniroyal pretreatment building on August 30, 1979. The capability of the bench scale systems to meet the MOE objectives under various operating conditions was evaluated during the period September 24, 1979 - July 31, 1980.

Continuous, 24 hour samples of primary clarified Town Sewage, and of pretreated Uniroyal wastewater were collected from the overflow of a pilot (90 gal.) clarifier fed by a Moyno pump connected to the Town flume and from the Uniroyal flume respectively. These composite samples were mixed together daily in the ratio of 2:1 (Town:Uniroyal), 5:1 and Town only for feed to Systems A, B, and C respectively. During the last six weeks of the study the C System was converted from Town only to another 5:1 system.

Figure 1 is a photograph of the overall treatability system in operation in Uniroyal's pretreatment building. Figure 2 is a schematic of one continuous flow section of the unit showing the flow rates and tank sizes for System A and B during Run 1. System C was the same with the exception of the aeration basin capacity and HRT which were 10.6 L and 12 hours, respectively, during Run 1.

A summary of the locations and parameters monitored during the study is presented in Table 3.

TABLE 3

NITRIFICATION TREATABILITY UNIT MONITORING

LOCATION	PARAMETERS MONITORED
Influents and Effluents	BOD_5, COD, SS, VSS, TKN, NH_3, pH, Alk, TP, Phenolics Conductivity
Aeration Basin	DO, MLSS, MLVSS, O_2 Uptake Rate.

2.3 Results

The average characteristics of the Elmira Town sewage, after primary clarification, and the pretreated Uniroyal wastewater during the study is presented in Table 4.

From these data it is clear that the Uniroyal wastewater contributed a large percentage of the oxygen demanding substances (i.e. BOD_5 and nitrogen).

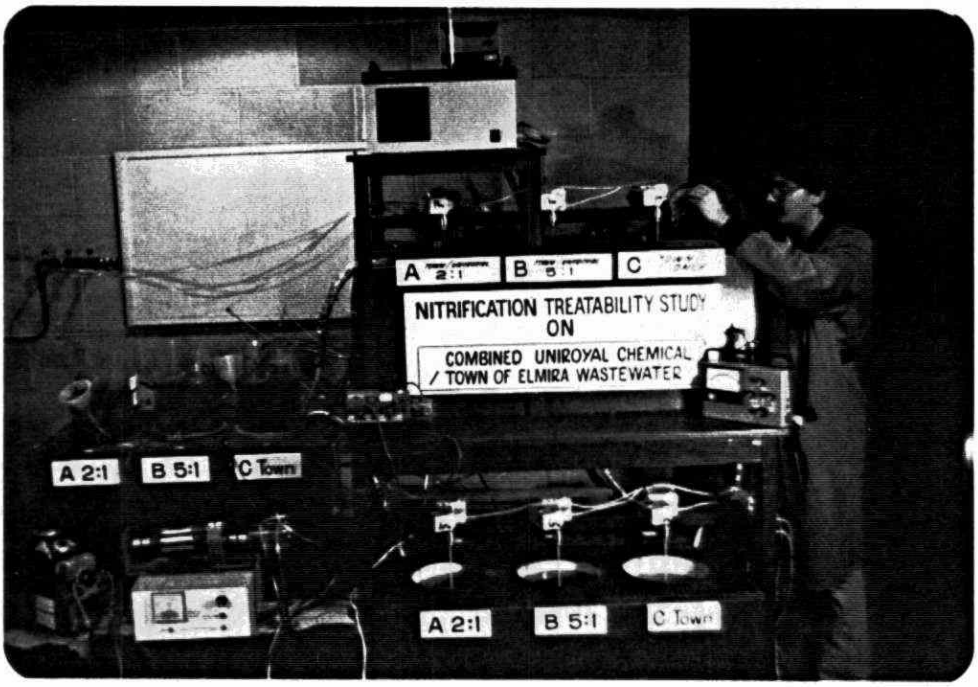

A 2:1
B 5:1
C
NITRIFICATION TREATABILITY STUDY
ON
COMBINED UNIROYAL CHEMICAL
/ TOWN OF ELMIRA WASTEWATER
A 2:1
B 5:1
C Town
A 2:1
B 5:1
C Town

FIGURE 1 PHOTOGRAPH OF CANVIRO NITRIFICATION TREATABILITY UNIT

TABLE 4. TOWN OF ELMIRA AND UNIROYAL WASTEWATER CHARACTERISTICS DURING NITRIFICATION TREATABILITY STUDIES (Sept/79 - July/80)

	Town	Uniroyal
TKN (mg/L)	30	104
NH_3 (mg/L)	15	85
BOD_5 (mg/L)	117	420
SS (mg/L)	94	156
pH	7.7	7.6
Alkalinity (mg/L)	367	337
Conductivity (umhos/cm)	2,374	42,113
Phenolics (ug/L)	10	970
Flow (mgd)	0.69	0.12

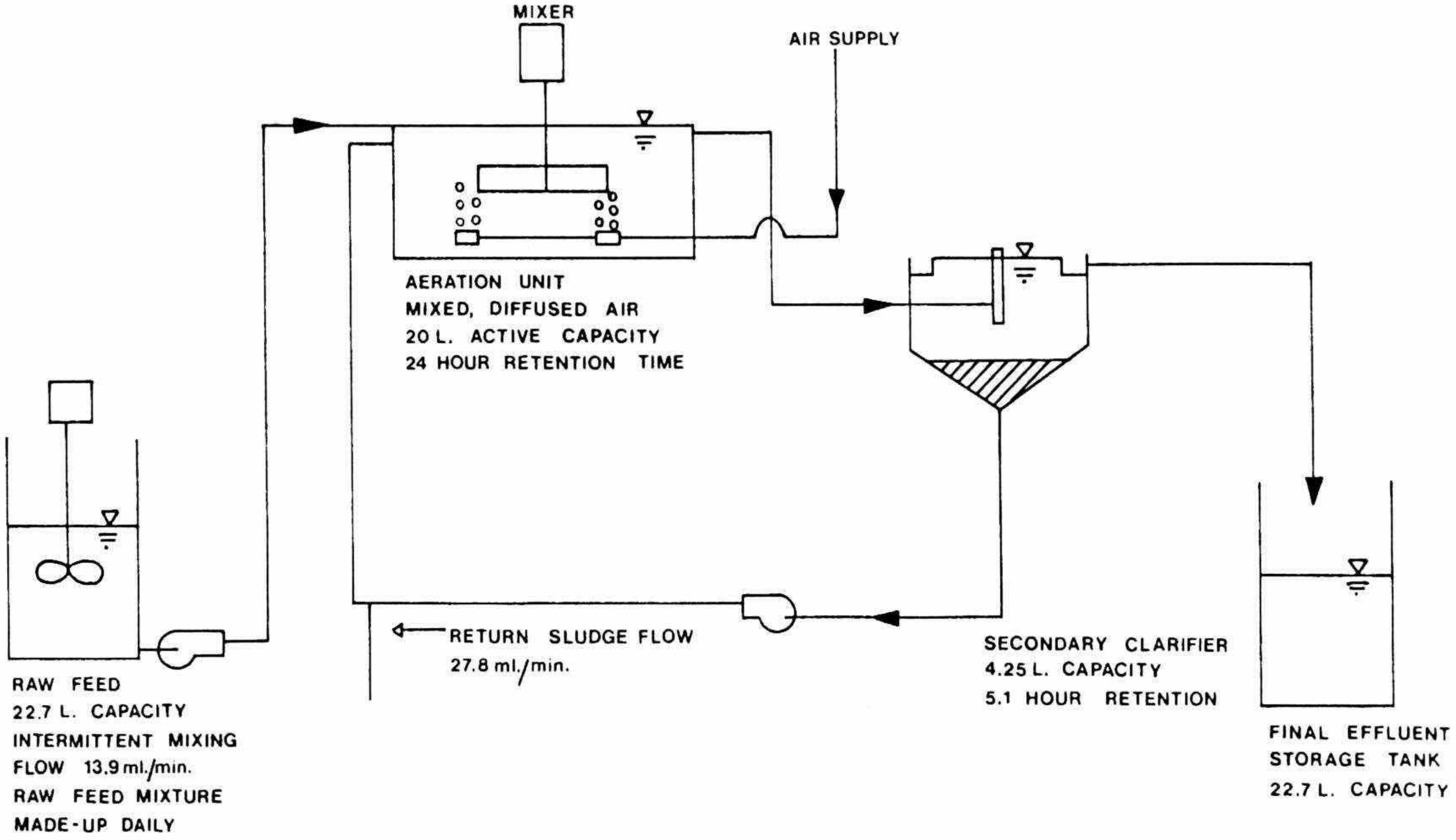


FIGURE 2 — SCHEMATIC DIAGRAM OF LABORATORY SCALE BIOLOGICAL NITRIFICATION TREATABILITY UNIT

The relatively high phenolics concentrations and conductivity of the Uniroyal wastewater were initially expected to present problems in the biological nitrification process. It is important to realize that the Uniroyal wastewater characteristics varied considerably throughout the various runs of the study and this factor had to be taken into account when evaluating the performance data. For example Figure 3 and Figure 4 illustrate the variability in conductivity and TKN, respectively, in the Town and Uniroyal wastewaters during the nitrification treatability study.

As outlined in Table 2 the experimental program was divided into individual treatment runs at selected operating conditions to optimize (i.e. minimize) HRT and SRT at the three flow ratios. The study culminated with a demonstration run at the selected optimum conditions under variable loadings of BOD and phenolics to verify the ability of the systems to recover from shock loadings and upsets.

The specific objectives, operating conditions, and general comments on each of the six runs are summarized in Tables 5 to 10. Tables and plots of influent and effluent data were prepared for all parameters at each of the three flow ratios during each run. Presentation of all the data is beyond the scope of this paper. However, two example plots of influent and effluent BOD and phenolics concentrations (for the 5:1 Town:Uniroyal System) are shown as Figures 5 and 6, respectively. The ability of the systems to recover rapidly from excessive loadings of BOD_5 and phenolics was effectively demonstrated and the overall conclusion of the nitrification treatability study was that the combined Town of Elmira/Uniroyal Chemical wastewater is treatable.

Specific conclusions included:

1. Nitrification - Town/Uniroyal wastewater requires:
   HRT = 21.6 hr.
   SRT = 25 days (Summer)
   = 50 days (Winter)
   - Town sewage alone requires:
   HRT = 7.2 hr.
   SRT = 10 days (Summer)
   = 20 days (Winter)

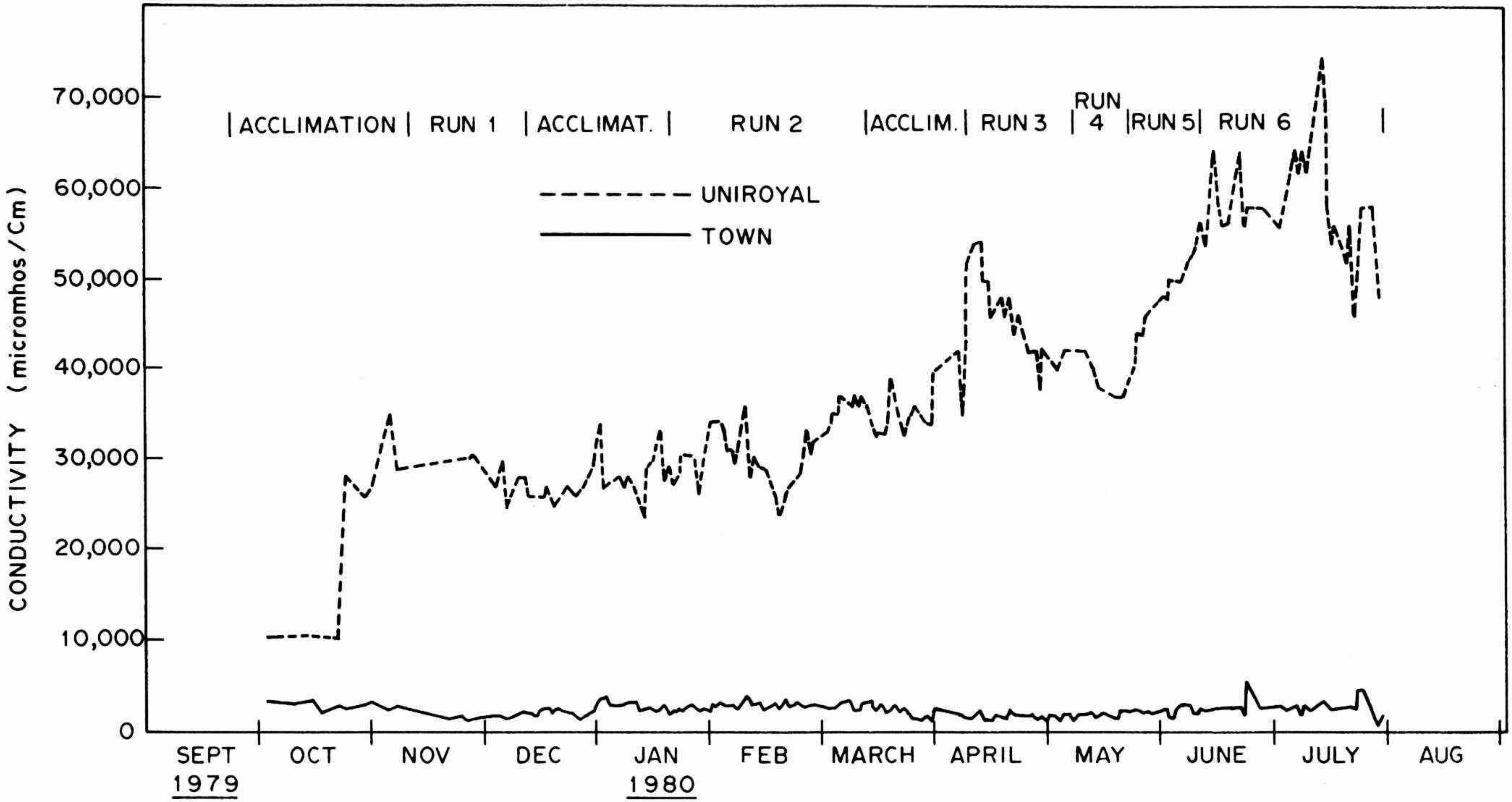


FIGURE 3 — CONDUCTIVITY VARIABILITY IN TOWN AND UNIROYAL WASTEWATERS DURING THE TREATABILITY STUDY

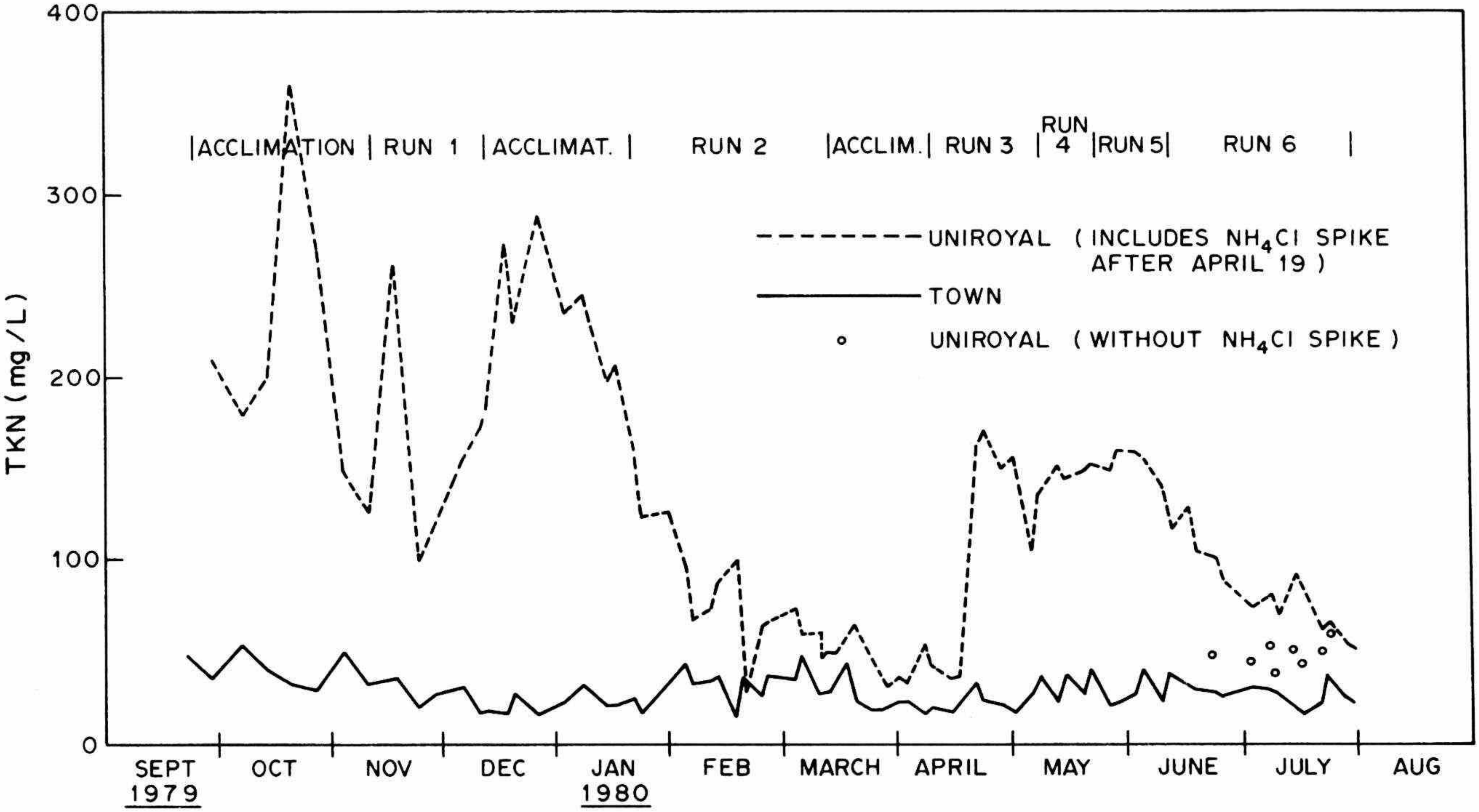


FIGURE 4 — TKN VARIABILITY IN TOWN AND UNIROYAL WASTEWATERS DURING THE TREATABILITY STUDY

TABLE 5. RUN NO. 1 RESULTS

OBJECTIVE: DETERMINE IF THE COMBINED WASTEWATER IS TREATABLE UNDER SUMMER CONDITIONS AT A REASONABLE DESIGN LEVEL, (I.E., PLANT SIZE AND COST).

	OPERATING CONDITIONS					TREATABILITY			
SYSTEM	HRT HRS	SRT DAYS	TEMP °C	F/M	BOD LOADING $LB/D/1000\ FT^3$	TKN (MG/L)	NH_3 (MG/L)	BOD (MG/L)	PHENOLICS (UG/L)
A 2:1	24	30	20	0.14	10.8	NO	YES	YES	NO
B 5:1	24	30	20	0.08	7.8	YES	YES	YES	NO
C TOWN	12	10	20	0.09	10.4	YES	YES	YES	YES

COMMENTS: PHENOLICS COULD BE A PROBLEM IN THE COMBINED TOWN/UNIROYAL TREATMENT SYSTEMS.

TABLE 6. RUN NO. 2 RESULTS

OBJECTIVE: DETERMINE IF THE WASTEWATER IS TREATABLE UNDER WINTER CONDITIONS AT A REASONABLE COST.

	OPERATING CONDITIONS					TREATABILITY			
SYSTEM	HRT HRS	SRT DAYS	TEMP °C	F/M	BOD LOADING $LB/D/1000\ FT^3$	TKN (MG/L)	NH_3 (MG/L)	BOD (MG/L)	PHENOLICS (UG/L)
A 2:1	24	45	8.0	0.11	18.4	YES	YES	YES	NO •
B 5:1	24	45	8.1	0.08	13.4	YES	YES	YES	NO •
C TOWN	12	22	8.1	0.12	15.9	YES	YES	YES	YES

COMMENTS: • HIGH AND VARIABLE PHENOLICS IN UNIROYAL WASTE IN THIS PERIOD (I.E., 50-11,000 UG/L). NITRIFICATION AND BOD REQUIREMENTS CAN BE MET BUT PHENOLICS ARE A PROBLEM AT HIGH INFLUENT CONCENTRATIONS.

TABLE 7. RUN NO. 3 RESULTS

OBJECTIVE: IDENTIFY MINIMUM HRT (WINTER).

	OPERATING CONDITIONS					TREATABILITY			
SYSTEM	HRT HRS	SRT DAYS	TEMP ^{o}C	F/M	BOD LOADING LB/D/1000 FT^3	TKN (MG/L)	NH_3 (MG/L)	BOD (MG/L)	PHENOLICS (UG/L)
A 2:1	12	47	5.2	0.07	22.3	NO	NO	NO	NO
B 5:1	12	50	5.6	0.06	18.2	YES	YES	YES	YES
C TOWN	6	25	6.2	0.15	27.8	YES	YES	YES	YES

COMMENTS: 2:1 SYSTEM MINIMUM HRT IDENTIFIED.
UNIROYAL WASTEWATER WAS WEAKER THAN "NORMAL", (PHENOLICS = 140-3400 UG/L).
5:1 SYSTEM MET ALL EFFLUENT OBJECTIVES.
AMMONIUM SPIKING TO THE UNIROYAL WASTEWATER WAS STARTED BECAUSE IT WAS SO WEAK.

TABLE 8. RUN NO. 4 RESULTS

OBJECTIVE: LOWER HRT ON TOWN AND 5:1 AGAIN.
RUN 2:1 AT ITS MINIMUM FEASIBLE HRT.

	OPERATING CONDITIONS					TREATABILITY			
SYSTEM	HRT HRS	SRT DAYS	TEMP ^{o}C	F/M	BOD LOADING LB/D/1000 FT^3	TKN (MG/L)	NH_3 (MG/L)	BOD (MG/L)	PHENOLICS (UG/L)
A 2:1	18	48	4.9	0.04	10.4	YES	YES	YES	**
B 5:1	8	48	5.6	0.08	18.8	NO	*	YES	NO
C TOWN	4	23	6.6	0.13	26.2	YES	YES	YES	YES

COMMENTS: TOWN SYSTEM STILL MET EFFLUENT OBJECTIVES BUT BULKING SLUDGE INDICATED HRT LIMIT WAS REACHED.
5:1 SYSTEM HRT LIMIT EXCEEDED. * INDICATES NH_3 ON VERGE OF BREAKTHROUGH.
** INDICATES 2:1 SYSTEM EFFLUENT PHENOLICS O.K. BY MOE ANALYSES (SLIGHTLY OVER OBJECTIVE BY CANVIRO ANALYSES).
UNIROYAL PHENOLICS LESS THAN 1000 UG/L IN THIS PERIOD.

TABLE 9. RUN NO. 5 RESULTS

OBJECTIVE: DETERMINE LIMITING SRT AT THE LIMITING HRT FOR EACH WASTEWATER MIX.

	OPERATING CONDITIONS					TREATABILITY			
SYSTEM	HRT HRS	SRT DAYS	TEMP °C	F/M	BOD LOADING LB/D/1000 FT^3	TKN (MG/L)	NH_3 (MG/L)	BOD (MG/L)	PHENOLICS (UG/L)
A 2:1	18/24	38.6	5.8	0.07	14.4	NO	YES	YES	NO
B 5:1	12	41.0	5.4	0.09	17.8	NO	NO	YES	NO
C TOWN	6	15.0	6.0	0.16	28.7	YES	YES	YES	YES

COMMENTS: TOWN SRT LIMITATION IDENTIFIED - 20 DAYS.
5:1 AND 2:1 SYSTEM SRT LIMITATION = 40 DAYS.
THIS WAS A NON-STEADY STATE PERIOD SO SOME AVERAGE EFFLUENT VALUES EXCEEDED EFFLUENT OBJECTIVES IN THE COMBINED SYSTEMS.

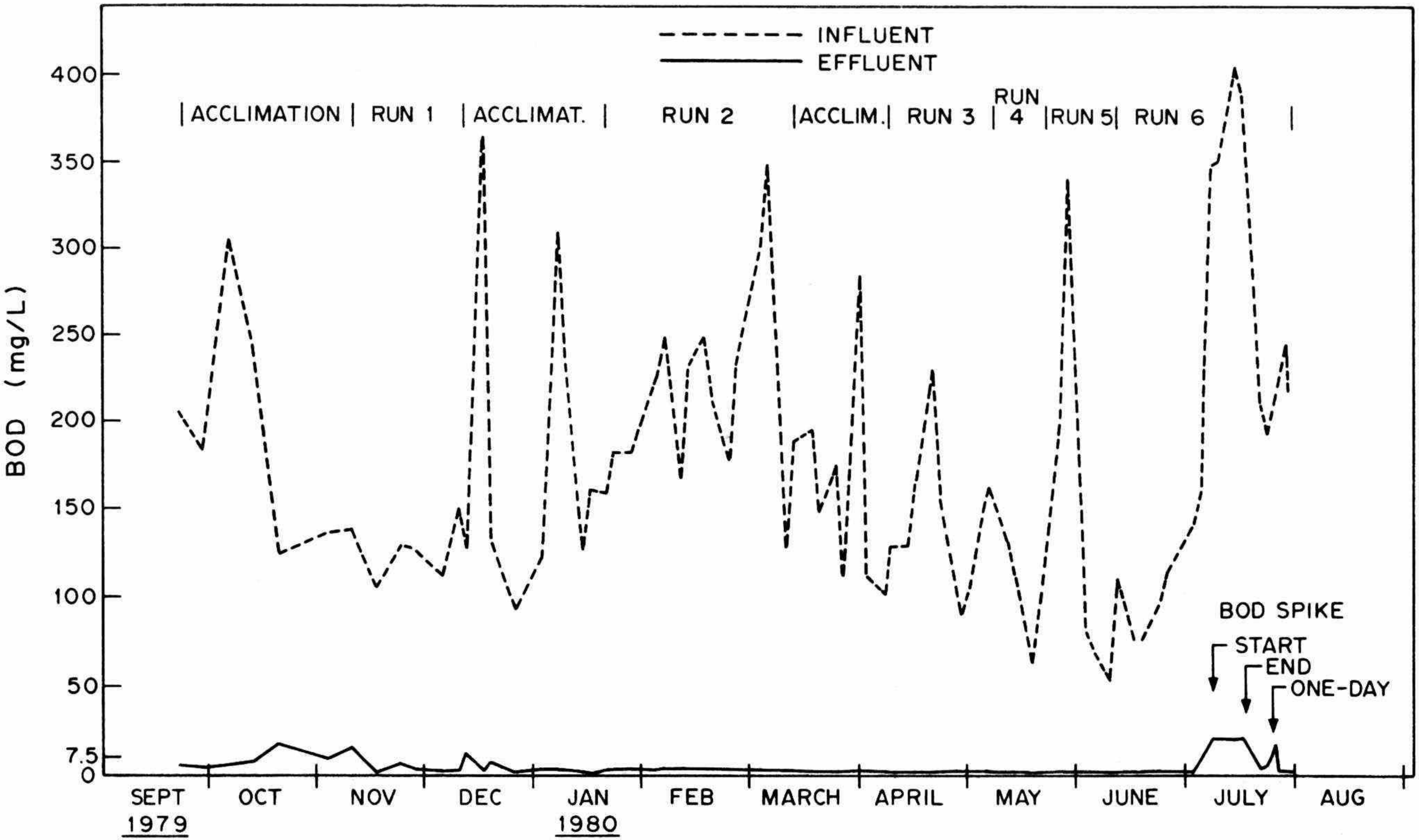
TABLE 10. RUN NO. 6 RESULTS

OBJECTIVE: TEST THE 5:1 SYSTEM UNDER VARIABLE BOD AND PHENOLICS LOADINGS AT WINTER DESIGN CONDITIONS.
TEST THE 2:1 SYSTEM UNDER VARIABLE BOD AND PHENOLICS LOADINGS AT SUMMER DESIGN CONDITIONS.

	OPERATING CONDITIONS					TREATABILITY			
SYSTEM	HRT HRS	SRT DAYS	TEMP °C	F/M	BOD LOADING LB/D/1000 FT^3	TKN (MG/L)	NH_3 (MG/L)	BOD (MG/L)	PHENOLICS (UG/L)
A 2:1	36	27.2	23.8 (8.1)	0.17	13.8	NO*	YES	YES	NO*
B 5:1	18	53.5	5.2	0.10	18.4	NO*	NO*	YES	NO*
C TOWN (5:1) **	18	23.6	5.7	0.11	16.8	NO*	YES	YES	NO*

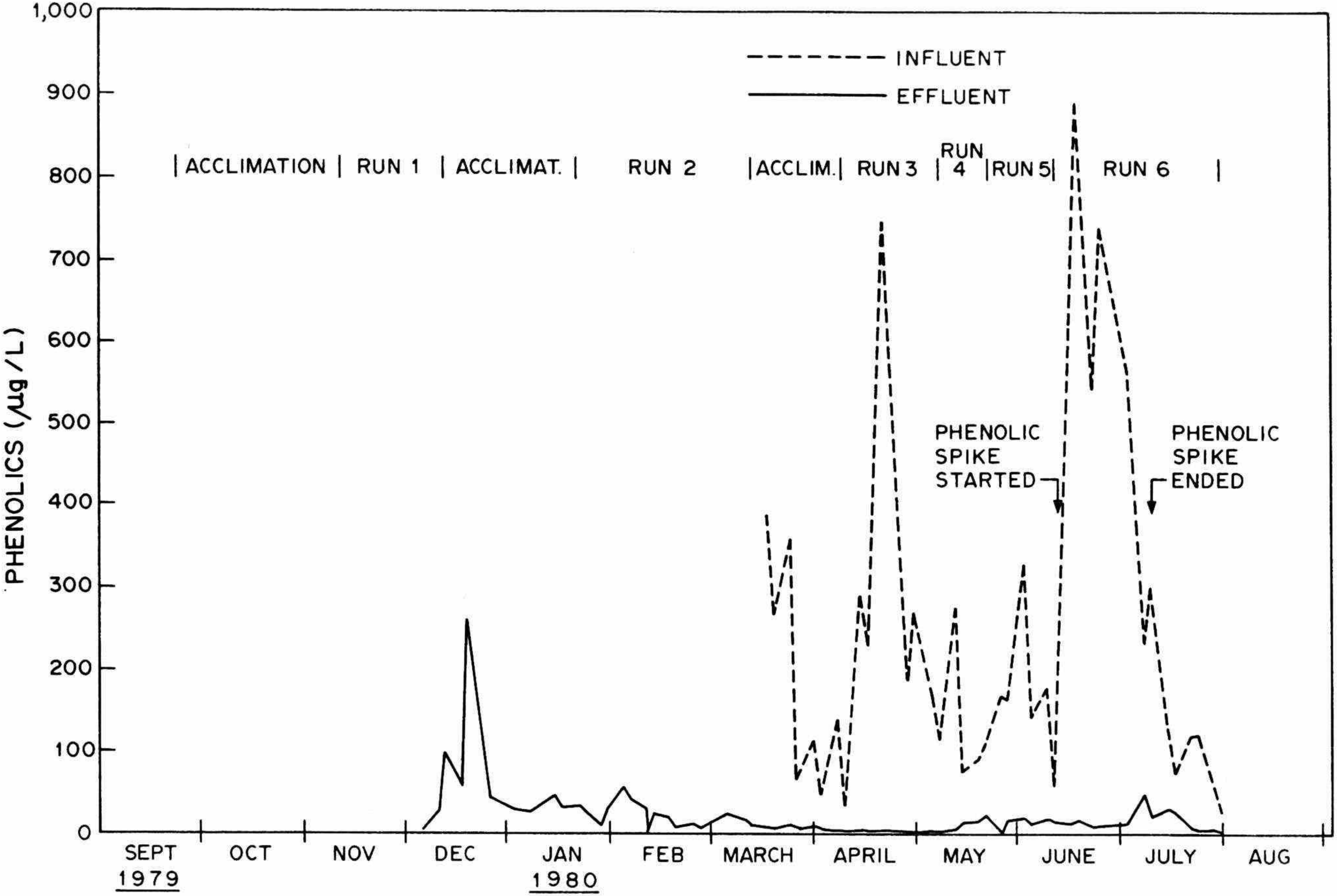
COMMENTS: * BECAUSE OF HIGH SHOCK LOADINGS OF BOD AND PHENOLICS AND GENERAL UPSET CONDITIONS, AVERAGE VALUES FOR SEVERAL PARAMETERS EXCEEDED THE OBJECTIVES.
HOWEVER, BOTH THE 2:1 (SUMMER) AND 5:1 (WINTER) SYSTEMS DEMONSTRATED THE ABILITY TO RECOVER RAPIDLY FROM EXCESSIVE BOD AND PHENOLICS SHOCK LOADING.

** SYSTEM C CONVERTED TO 5:1 DURING RUN 6



ELMIRA WPCP TREATABILITY STUDY

FIGURE 5 — SYSTEM B (5:1) INFLUENT AND EFFLUENT BOD CONCENTRATIONS



ELMIRA WPCP TREATABILITY STUDY
FIGURE 6 — SYSTEM B (5:1) INFLUENT AND EFFLUENT PHENOLICS CONCENTRATIONS

2. BOD_5 - effluent objective should be readily achieved at the design conditions for nitrification.
   5:1 and 2:1 systems recovered rapidly from shock loads equivalent to over 4,000 lbs. BOD/day.

3. Phenolics - effluent objective should be achieved if combined influent phenolics concentration is less than 500 ug/L.

4. TP & SS - should be no problem since effluent filtration is provided.

5. pH - pH control will continue to be required at the Uniroyal pretreatment building.
   Lime addition will be required if the combined influent NH_3 concentration exceeds 30 mg/L (because of alkalinity loss during nitrification).

The treatability study results were used directly for process design of the Elmira WPCP. In summary the major process design requirements for the expanded/upgraded plant to meet the MOE effluent guidelines were:

1. Aeration basin increase = 600,000 gallons
2. Aeration capacity increase = 5860 lb O_2/day
3. Sludge return increase = 416 GPM
4. Sludge wasting requirement = 2-20 GPM
5. Secondary clarifier S.A. increase = 6360 ft.

In addition, the treatability results from the Town Only system versus the results from the 5:1 (average Town:Uniroyal ratio) system formed the technical basis for the development of capital and operating cost sharing agreements between the Regional Municipality of Waterloo and Uniroyal Chemical Ltd.

3 ANAEROBIC DIGESTION TREATABILITY STUDY

3.1 Objectives

After conducting a preliminary investigation of disposal alternatives for the Elmira WPCP, the most feasible methods were identified as:

1. Anaerobic digestion - farmland utilization
2. Lime conditioning/stabilization - filter press dewatering - landfill disposal of WAS: with primary sludge hauling to another regional WPCP for digestion
3. Lime conditioning/stabilization - filter press dewatering - landfill disposal of all sludge produced.

Because the existing anaerobic digesters at the Elmira WPCP had not been functional for several years, it was considered necessary to conduct a treatability study.

In June 1979, Canviro Consultants Ltd. was authorized to proceed with anaerobic digestion studies to determine the technical and economic feasibility of using the existing digesters at the Elmira Water Pollution Control Plant to digest the primary and waste activated sludge produced in the proposed, expanded/upgraded plant.

Specific objectives included:

1. To estimate the quantity and quality of raw digested sludge that will be produced in the expanded plant.
2. To determine whether any substances are present in either the primary or waste activated sludge that prove to be toxic or inhibitory to anaerobic digestion.
3. To prepare an estimate of digester operating costs for the expanded plant.
4. To make recommendations on the future use of the anaerobic digesters at the Elmira WPCP.

3.2 Methodology

The overall experimental design of the study is shown in Table 11. The factors investigated were:

1. Solids Retention Time (SRT).
2. Volatile Solids Loading Rate.
3. Ratios of Town Primary Sludge:Combined Town/Uniroyal Waste Activated Sludge.
4. Temperature (controlled to 95°F; 35°C).

TABLE 11. EXPERIMENTAL DESIGN FOR ANAEROBIC DIGESTION STUDY AT THE ELMIRA, WPCP

	RUN 1 (8 wks) ACCLIMATIZATION AND PRELIMINARY	RUN 2 (8 wks) LONGER TERM STEADY STATE	RUN 3 EXTENDED TROUBLE SHOOTING
NOMINAL REACTOR CONDITIONS	0.05 lb VS/ft^3 26 day SRT	Same as RUN 1	NR
Reactor			
A	Primary:WAS (3:1)	Same as RUN 1	NR
B	Primary:WAS (6:1)	Same as RUN 1	NR
C	Town Primary Only	Same as RUN 1	NR

NR = Run was not required because no problems were encountered.

The program was designed such that if no toxicity or operational problems were encountered in Run 1, then only one more Run would be required to evaluate longer term stability and develop methane production data for an economic assessment of the process at this plant. No toxicity was encountered in Run 1 so trouble-shooting was not necessary. Run 2 was essentially an extension of Run 1 conducted at the same conditions to develop longer term operating and economic data.

A temperature controlled system with three 22 litre batch fill-and-draw stainless steel reactors, fitted with biogas collection piping was fabricated and moved onto the site. A photograph of the unit is shown in Figure 7. A 90 gal. pilot clarifier was installed in the basement of the Uniroyal pretreatment building. The clarifier was piped into the Town flume to provide primary settling of Town raw sewage. This was necessary to produce a Town Only primary sludge for the digester study and also a clarified Town Only sewage for the nitrification study. The clarifier was continuously fed using a Moyno pump calibrated to 1.0 gpm. This yielded a clarifier overflow rate of 488 Igpm/ft^2.

Table 12 summarizes the sampling and analysis performed during the study.

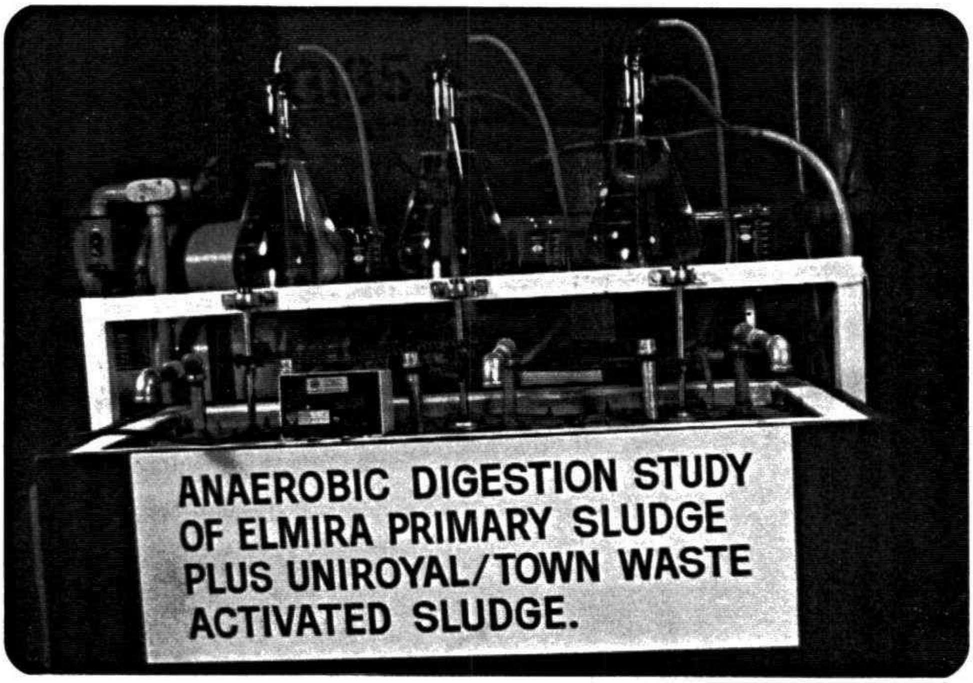


FIGURE 7. PHOTOGRAPH OF ANAEROBIC DIGESTION TREATABILITY UNIT, ON-SITE AT THE ELMIRA WPCP.

TABLE 12. ANAEROBIC DIGESTER MONITORING

RAW PRIMARY + WAS	TS 2-d comp. VS 2-d comp. pH Daily comp.
DIGESTED SLUDGE	TS, VS, pH; Same Vol. Acids 2/wk. Alkalinity 2/wk.
DIGESTER GAS	CH_4 Content - Weekly Volume - Daily
SUPERNATANT & EFFLUENT	BOD, TKN, NH_3, Metals TP, COD; 2 times
OTHER	Trace organics in another study funded by the Ministry of the Environment and Environment Canada.

3.3 Results

The average characteristics of the Elmira primary and WAS characteristics during the steady state test period are presented in Table 13. One of the more noteworthy observations was the relatively low percentage of volatile solids in the WAS because of the high salt content in the Uniroyal wastewater. Numerous plots and tables of data were generated for each of the relevant process operational parameters over the study period. For this paper, data presentation is limited to summary tables of digested sludge characteristics (Table 14) and performance data (Table 15) from the steady state period to illustrate the type of data collected and to give an overview of the study results. All three digesters achieved good volatile solids reduction (67-70%) and biogas production (13.0 - 14.7 ft^3/lb VS added) and exhibited good process stability over the range of conditions investigated. Based on scale-up calculations, the mixing capability of the existing primary digester was determined to be adequate.

A detailed energy balance and economic assessment of anaerobic digestion at the Elmira WPCP estimated the operating costs for heating the digesters and control buildings to be between $625.00 - $2,000.00 per year, depending on the degree of sludge thickening achieved in the expanded, upgraded plant. (The 1979 cost of heating the buildings alone was $1,126.00).

The overall conclusion of the anaerobic digestion treatability study was that digestion of the Elmira WPCP sludge would be technically and economically feasible. However two major concerns arose during the study regarding the utilization of Elmira WPCP sludge on agricultural land.

1. Low ammonia: mercury ratio may not meet MOE sludge utilization guidelines over the long term.
2. A Federal/Provincial study at the Elmira WPCP identified several potentially toxic trace organic contaminants in the wastewater and sludge streams, raising concerns about the sludge suitability for farmland utilization.

In view of these concerns it was recommended that sludge dewatering-landfill disposal alternatives be fully evaluated before giving further consideration to digestion followed by disposal on farmland.

TABLE 13. ELMIRA PRIMARY AND WAS CHARACTERISTICS DURING RUN 2 (DEC. 1, 1979 - JAN. 31, 1980)

		pH	TS (% wt)	VS (% of Total)
PRIMARY SLUDGE	- Average	5.8	2.94	74.5
	- Range	5.2-6.5	1.02-4.41	64.0-82.5
WASTE ACTIVATED SLUDGE	- Average	7.1	1.33	55.3
	- Range	5.8-7.6	0.99-2.72	37.3-65.6

TABLE 14. DIGESTED SLUDGE CHARACTERISTICS DURING RUN 2 STEADY STATE PERIOD

DIGESTER	pH	TS (%)	VS (% of Total)	ALKALINITY (mg/L)	VOLATILE ACIDS (mg/L)
A PRIMARY:WAS = 3:1					
AVERAGE	6.8	1.09	53.6	2,570	510
RANGE	6.1-7.6	0.63-1.36	49.4-58.1	2,000-4,050	170-965
B PRIMARY:WAS = 6:1					
AVERAGE	7.0	1.18	56.7	2,710	495
RANGE	6.4-7.5	1.02-1.37	45.4-62.6	2,500-3,090	175-850
C PRIMARY ONLY					
AVERAGE	7.0	1.15	58.5	2,730	405
RANGE	6.3-7.4	1.02-1.32	51.1-63.6	2,520-3,230	160-765

TABLE 15. PERFORMANCE SUMMARY FOR RUN 2 STEADY-STATE PERIOD

DIGESTER	RETENTION TIME (DAYS)	DIGESTER LOADING RATE (LB VS/Ft^3/D)	RAW SLUDGE		DIGESTED SLUDGE		OVERALL PERFORMANCE		
			TS (%)	% VS	TS (%)	% VS	VOL SOLIDS REDUCTION (%)	GAS PROD'N FT^3/1000 FT^3/D	SPECIFIC GAS PROD'N FT^3/LB VS ADDED
A PRIMARY WAS 3:1	26	0.044	2.55	72	1.09	53.6	68	626	14.0
B PRIMARY WAS 6:1	26	0.048	2.73	73	1.18	56.7	67	687	13.0
C PRIMARY ONLY	26	0.054	2.94	75	1.15	59	70	855	14.7

4 SLUDGE DEWATERING STUDY

4.1 Objectives

The overall objective of the sludge dewatering study was to develop a design and cost estimate for a system to produce an acceptable sludge for disposal at a Region of Waterloo landfill site. Because of the requirements of the Regional municipality for a minimum sludge cake content of 30% TS, as well as stringent odour and leachate quality criteria, it was apparent that lime conditioning/ stabilization followed by dewatering by plate-and-frame filter press would be necessary.

The sludge dewatering study was designed in two parts:

1. Laboratory Scale Tests: - to determine dosages of lime and ferric chloride required to condition (and stabilize) the sludge prior to dewatering. (i.e. select dosages for pilot testing.)
2. Pilot Scale Tests: - to develop process design and cost information (i.e. sizing of filter press, pumps, conditioning tanks, etc.)

4.2 Laboratory Scale Tests

To evaluate the effect of various combinations of sludge types and sludge age on dewaterability, three types of sludges were used for the laboratory characterization and analysis:

1. Partially digested primary plus waste activated sludge collected from the existing sludge storage (digester) tanks.
2. Fresh primary from the pilot scale clarifier setup on site, combined with waste activated sludge from the sludge return line. The waste activated sludge was thickened to 3.4% (total solids) prior to being combined with the primary sludge in the ratio of 45% and 55% primary (by volume).
3. Waste activated sludge was collected from the sludge return line and thickened to 3.6% (total solids) prior to testing.

Conditioning was performed on 1,000 millilitre samples of each sludge type using lime plus ferric chloride dosages ranging from controls with no chemical addition to dosages as high as 90% lime alone, and 30% lime plus 7% ferric chloride.

Specific resistance to filtration determinations were performed on the control sludge and each of the conditioned samples for all three sludges.

Tables 16, 17, and 18 present the effects of chemical conditioning on each of the three sludges tested. Sludge feed solids and cake solids are also included. Cake solids tests can not be directly related to full scale pressure filtration performance and should only be compared relative to each other. The pH of the initial sludge prior to conditioning, and after 20 minutes of flocculation is included in each table. A final pH of 12.0 or greater than after 20 minutes, was the minimum acceptable pH that could be considered as "stabilization".

Generally, a specific resistance value of approximately 0.1 x 10^{13} m/kg or less is required in order for filtration to be an efficient method of dewatering. The partially digested primary plus waste activated sludge showed the least improvement with chemical addition and the poorest dewatering characteristics. Fresh primary plus waste activated sludge, pumped to the digester tank for storage, may be stored for long periods from time to time as a result of mechanical problems, or other operational problems. This storage will result in a partially digested, difficult to dewater sludge. To assure that the full scale facility for all the sludge would be designed for the worst case, the partially digested combined sludge was selected over the fresh combined sludge for pilot scale testing. It was decided to evaluate two levels of conditioning in the pilot runs (i.e., 30% lime + 7% $FeCl_3$ and 20% lime + 5% $FeCl_3$ which gave specific resistances of 0.11 x 10^{13} m/kg, and 0.34 x 10^{13} m/kg; both had final pH greater that the 12.0 required for stabilization).

For sizing facilities for WAS only, dosages of 30% lime + 7% $FeCl_3$ were selected for pilot testing. This combination resulted in a specific resistance of 0.074 x 10^{13} m/kg and a pH of 12.2 in the lab tests.

4.3 Pilot Scale Tests

Four pilot scale filter press dewatering runs were performed on partially digested primary plus waste activated sludge from the existing digester tank. These runs were aimed at developing design information for dewatering all the combined sludge.

A Sperry 18 inch x 18 inch square, plate and frame pressure filter operating at 80-125 psig was used for the pilot scale tests. A polypropylene/cotton weave cloth was used as a filter media for all but one test where a single weave nylon cloth was used.

Sludge was obtained in 45 Igal drums and transported to the pilot unit which was located in Burlington. For each run, approximately thirty-five gallons

TABLE 16

CHEMICAL CONDITIONING EFFECTS ON SPECIFIC RESISTANCE TO FILTRATION FOR AGED PRIMARY AND WASTE ACTIVATED SLUDGE, ELMIRA STP.

CONDITIONING % HIGH CALCIUM LIME	 % FERRIC CHLORIDE	FEED SOLIDS % (BEFORE CONDITIONING)	CAKE SOLIDS %	INITIAL pH	FINAL pH	SPECIFIC RESISTANCE X 10^{13} m/kg
-	-	4.97	18.8	6.8	6.8	6.8
30	-	4.74	20.7	6.9	12.4	2.0
45	-	4.74	22.1	6.8	12.4	1.2
60	-	4.74	24.7	6.8	12.4	0.88
90	-	4.74	25.7	6.8	12.4	0.46
15	3	4.74	20.1	6.8	11.7	1.7
15	5	4.74	17.8	6.8	10.9	0.42
15	7	4.66	18.0	6.8	10.1	0.21
20	3	4.66	19.2	6.8	12.2	0.59
20	5	4.66	19.5	6.8	12.2	0.34
20	7	4.66	18.1	6.8	11.8	0.18
30	3	4.97	19.5	6.9	12.6	0.35
30	5	4.97	19.0	6.9	12.5	0.29
30	7	4.97	17.9	6.9	12.2	0.11

TABLE 17

CHEMICAL CONDITIONING EFFECTS ON SPECIFIC RESISTANCE TO FILTRATION FOR FRESH PRIMARY AND WASTE ACTIVATED SLUDGE, ELMIRA STP.

CONDITIONING		FEED SOLIDS % (BEFORE CONDITIONING)	CAKE SOLIDS %	INITIAL pH	FINAL pH	SPECIFIC RESISTANCE X 10^{13} m/kg
% HIGH CALCIUM LIME	% FERRIC CHLORIDE					
–	–	4.8	13.7	6.2	6.2	16.0
30	–	4.8	25.3	5.9	12.3	0.65
45	–	4.8	24.2	5.9	12.4	0.65
60	–	4.8	29.1	5.9	12.3	0.34
90	–	4.8	24.9	5.9	12.3	0.44
15	3	4.8	23.0	5.9	12.0	0.095
15	5	4.7	22.2	5.7	11.4	0.073
15	7	4.7	21.2	5.7	11.4	0.050
20	3	4.7	20.7	5.7	12.2	0.13
20	5	4.7	21.3	5.7	12.1	0.052
20	7	4.7	21.1	5.7	12.0	0.032
30	3	4.8	23.5	6.2	12.2	0.070
30	5	4.8	23.1	6.2	12.2	0.053
30	7	4.8	20.7	6.2	12.2	0.034

TABLE 18

CHEMICAL CONDITIONING EFFECT ON SPECIFIC RESISTANCE TO FILTRATION FOR THICKENED WASTE ACTIVATED SLUDGE, ELMIRA STP.

CONDITIONING		FEED SOLIDS % (BEFORE CONDITIONING)	CAKE SOLIDS %	INITIAL pH	FINAL pH	SPECIFIC RESISTANCE X 10^{13} m/kg
% HIGH CALCIUM LIME	% FERRIC CHLORIDE					
-	-	3.6	17.1	7.2	7.2	2.1
30	-	3.6	18.4	7.2	12.4	0.95
45	-	3.6	18.2	7.2	12.5	0.43
60	-	3.6	20.0	7.2	12.5	0.46
90	-	3.6	22.0	7.2	12.5	0.33
15	3	3.6	15.6	7.2	12.0	0.31
15	5	3.6	14.4	7.2	11.3	0.22
15	7	3.6	14.8	7.2	11.6	0.20
20	3	3.7	15.5	7.2	12.3	0.24
20	5	3.7	15.0	7.2	12.1	0.12
20	7	3.7	14.0	7.2	11.9	0.11
30	3	3.7	16.4	7.5	12.3	0.20
30	5	3.7	15.5	7.5	12.3	0.11
30	7	3.7	14.3	7.5	12.2	0.074
NT75A (10g/1)		3.7	19.7	7.5	7.6	1.1
20% + 20 g NT75A		3.7	19.7	7.5	12.4	0.63

of sludge was pumped into a conditioning tank, stirred and analyzed for total solids content of the sludge. Ferric chloride (at a total stock concentration of 10% as $FeCl_3$), was added to the sludge conditioning tank, followed immediately by the required volume of lime [10% as $Ca(OH)_2$]. Twenty minutes after chemical addition, the pH was measured and recorded.

The conditioned sludge was fed into two chambers of the pilot scale pressure filter unit immediately after the twenty minute sludge mixing cycle. Filtrate was collected and the times recorded to give a plot of total filtrate volume vs. time. A flow proportioned filtrate sample was collected and analyzed for pH, solids (total and suspended) and BOD_5. At the end of each run, the sludge volume consumed was recorded and a cake solids analysis was performed on the sludge cake. A complete cake from one chamber was kept from each filter run, for leaching tests as required.

Subsequently, four mini-filter press runs on a 5 inch x 5 inch pilot unit were performed using aged, waste activated sludge to verify design of the system for dewatering WAS only.

Unsaturated column leaching tests were used to estimate the leachability of the sludge under simulated landfill conditions. Samples of leachate from both a wet (cake solids as removed from the pressure filter) and a dry (cake sample dried at 103° overnight) leaching test were analyzed to determine leachate quality.

The four runs performed using aged primary + waste activated sludge were:

1. 20% Lime + 5% Ferric Chloride, 80-120 psig, poly/cotton weave and 1" plates.
2. 30% Lime + 7% Ferric Chloride, 80-120 psig, poly/cotton weave and 1" plate.
3. 30% Lime + 7% Ferric Chloride, 80-120 psig, poly/cotton weave and 2" plates.
4. 30% Lime + 7% Ferric Chloride, 80-120 psig, polypropylene cloth and 1 " plates.

The minimum performance criteria for filter press selection were:

1. Cycle times of less than 6 hours.
2. Cake solids greater than 30%.
3. Cake must pass leaching and odour tests for acceptability on landfill.

A plot of the filtrate volume vs. time for the pilot runs on combined sludge is shown in Figure 8. A summary of the chemical conditioning, filtration media, cake thickness, filtration solids, run lengths, and filtrate characteristics are shown in Table 19.

Based on the pilot scale testing, only the run using a one inch chamber and the poly/cotton weave cloth with chemical conditioning of 30% Lime and 7% ferric chloride met the minimum performance criteria. The cake from this run was used for the leaching test (which it passed) and the process design for the facility to dewater all the combined sludge was scaled up from this data.

Pilot testing was also carried out on the WAS alone and results of the optimum run were used to develop a process for a dewatering facility for the WAS only. It was determined that to meet the performance criteria for dewatering the WAS only the following conditions would be required:

- 3% ferric chloride + 60% lime
- 1 inch chambers
- poly/cotton weave cloth

From the dewatering test results a process design and cost estimates were developed for each of the two dewatering alternatives. As an example, Figure 9 is presented to show schematically the sludge dewatering and disposal system for handling the combined sludge at the Elmira WPCP.

Accurate capital and operating costs for the two sludge dewatering alternatives were made from the dewatering study results for comparison to costs for the anaerobic digestion alternative. After evaluation of all of the technical, economic and political aspects of the sludge disposal alternatives, the Region of Waterloo selected the alternative of filter press dewatering and landfilling the combined sludge.

Sludge production data from the sludge studies and from the nitrification study was utilized as the basis for a cost sharing agreement between the Region and Uniroyal for the sludge handling and disposal component of the expanded/upgraded Elmira WPCP.

5 SUMMARY OF TREATABILITY STUDY BENEFITS

The overall benefit of conducting treatability studies from a technical viewpoint is obtaining the assurance that the proposed treatment works will meet the effluent and sludge disposal objectives set out for the expanded, upgraded plant.

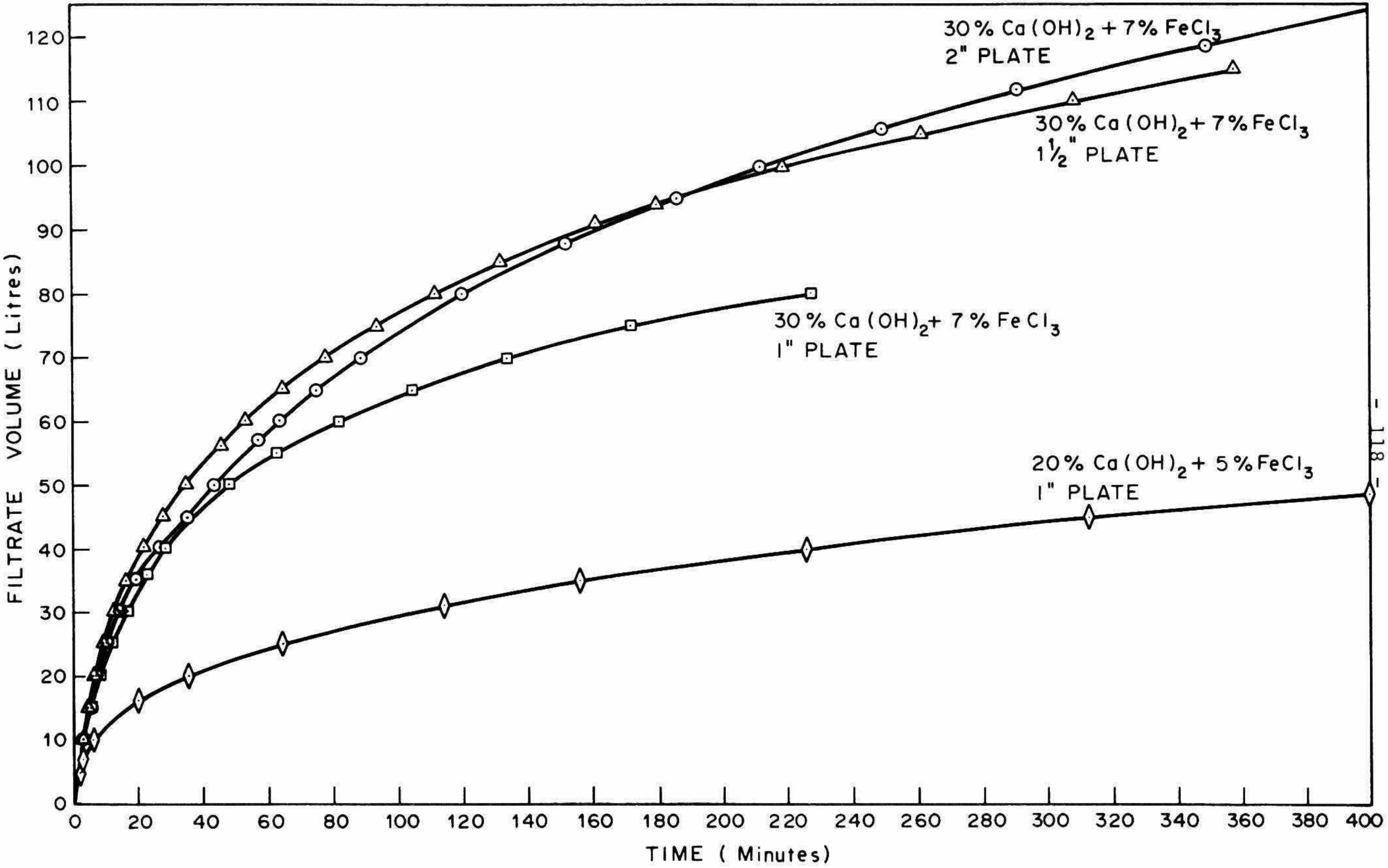


FIGURE 8—FILTRATE VOLUME COLLECTED VS. TIME FOR THE PILOT SCALE PRESSURE FILTER RUNS PERFORMED ON AGED ELMIRA PRIMARY + WASTE ACTIVATED SLUDGE

TABLE 19

PILOT SCALE PRESSURE FILTER RUNS ON AGED PRIMARY AND WASTE ACTIVATED SLUDGE, ELMIRA STP.

CONDITIONING		MEDIA	PLATE THICK- NESS	pH		TOTAL FEED (SLUDGE) SOLIDS (g/l)	TOTAL FEED SOLIDS AFTER COND. (g/l)	VOLUME OF SLUDGE USED (l)	TOTAL CAKE SOLIDS (%)	VOLUME OF FILTRATE (l)	FILTRATE TOTAL SOLIDS (mg/l)	FILTRATE SUSPENDED SOLIDS (mg/l)	FILTRATE COMPOSITE BOD_5 (mg/l)	RUN LENGTH (min)
$Ca(OH)_2$ %	$FeCl_3$ %			INITIAL	FINAL									
20	5	POLY/ COTTON	1"	6.8	11.8	52.4	60.61	50 l	20 to 25	48.8	8030	365	2189	400*
30	7	POLY/ COTTON	1"	6.8	12.1	49.3	60.25	88 l	35 to 37	80.0	12090	102	2163	228
30	7	POLY/ COTTON	2"	6.8	12.0	44.9	57.98	141 l	21 to 26	124.5	10870	303	1690	400*
30	7	POPR 801	1.5"	6.8	12.1	48.1	56.62	134 l	25.6 to 35.5	115.3	13240	660	1818	358+

* Run terminated after 400 minutes (6 2/3 hours).

+ Run terminated when feed solids appeared on the filtrate.

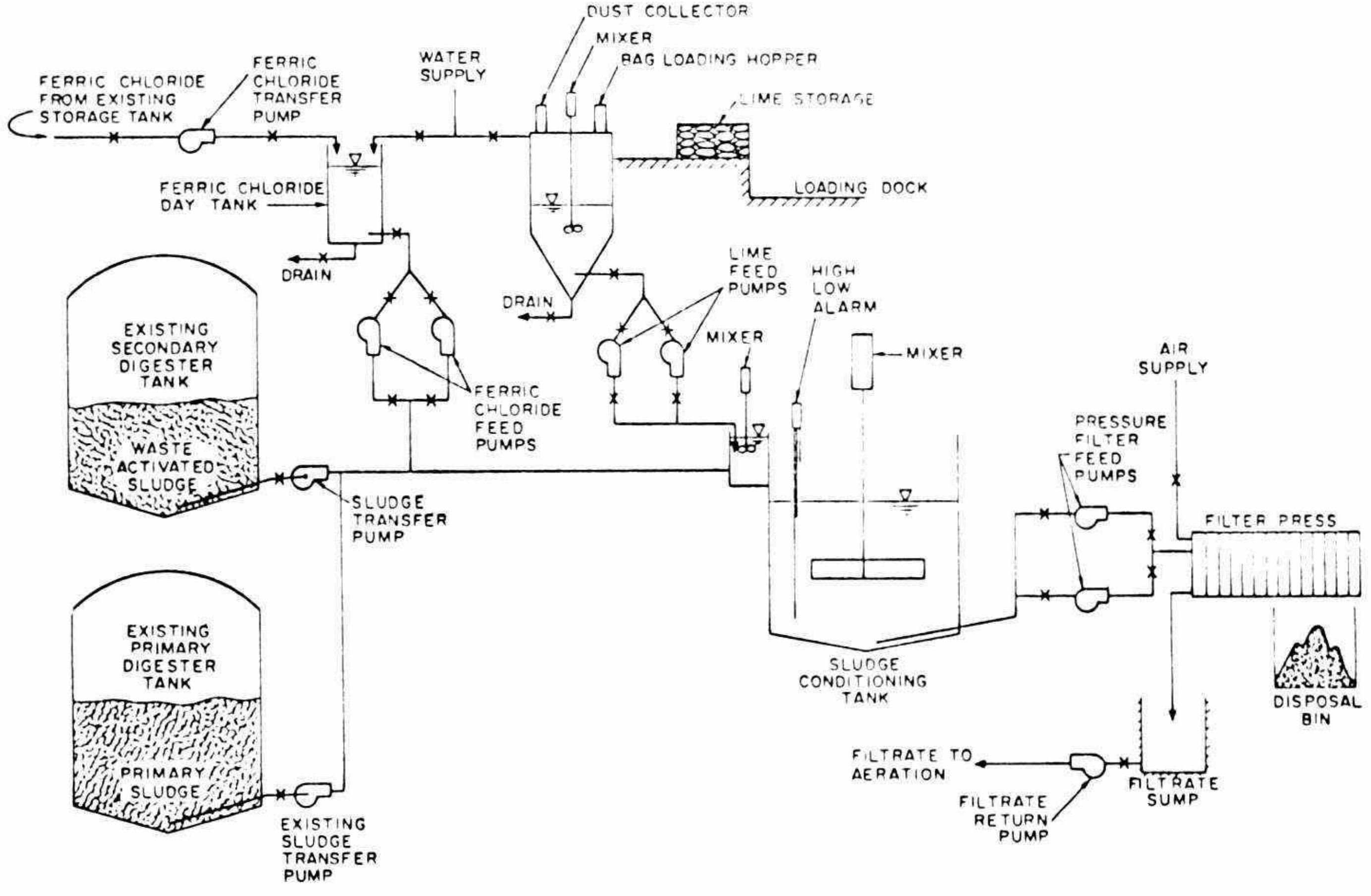


FIGURE 9 — SCHEMATIC OF SLUDGE DEWATERING FACILITY FOR THE COMBINED ELMIRA WPCP SLUDGE

The treatment facility size can be optimized/minimized to improve efficiency and reduce capital costs. Specifically, for the Elmira WPCP, the nitrification treatability study minimized the aeration basin size and final clarifier surface area based on the minimum HRT and SRT that achieved the effluent objectives in the study. The sludge production data generated in the nitrification study and sludge studies resulted in cost savings by allowing design of the minimum size dewatering facility to meet the landfilling criteria imposed by the Region of Waterloo.

In summary, it can be stated that treatability studies are particularily good for upgrading municipal/industrial plants (where the wastes are available, and where cost sharing is often involved).

Almost invariably, the cost savings by optimization and minimization of facility size, through reduction of design safety factors, outweighs the cost of treatability studies. The development of operational strategies for the expanded, upgraded plant and assurances that treatment objectives will be met are valuable additional benefits of treatability studies.

ACKNOWLEDGEMENTS

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The cooperation and assistance of Uniroyal Chemical Ltd. personnel throughout the project, is greatly appreciated, as is the assistance provided by the Ministry of the Environment.

"UPGRADING THE REGIONAL MUNICIPALITY OF HALTON'S MILTON WATER POLLUTION CONTROL PLANT TO MEET STRINGENT EFFLUENT DISCHARGE REQUIREMENTS"

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Presented at the

Pollution Control Association of Ontario -
Ministry of The Environment Seminar, Toronto

March 10, 1982

"UPGRADING THE REGIONAL MUNICIPALITY OF HALTON'S MILTON WATER POLLUTION CONTROL PLANT TO MEET STRINGENT EFFLUENT DISCHARGE REQUIREMENTS"

by: (1) D.R. Morrier, P.Eng., (2) P.J. Laughton, P.Eng., (3) S.N.Gergis, P.Eng.

The Regional Municipality of Halton's Milton Water Pollution Control Plant is one of six treatment facilities within the Region and is located approximately 58 kilometers west of Toronto with its geographical relationship to other centres as noted on Figure 1.

Sixteen Mile Creek traverses the Town of Milton and has played a major role in the development of effluent criteria for the 1978 Milton Treatment Plant expansion. Due to the sensitive nature of this Creek and on the basis of a proposed average daily plant flow of 12910 m^3/day, the Ontario Ministry of the Environment imposed the following effluent parameters:

- BOD_5 of 4.2 mg/L or 55 kg/day,
- Soluble Phosphorus of 0.43 mg/L or 5.5 kg/day,
- Chlorine residual of 0.08 mg/L,
- Nitrification 90% during warm weather.

These requirements resulted from an extensive water quality study carried out on the Creek in July 1973.

Resulting from field and laboratory pilot studies by Pollutech Limited, it was concluded that nitrification removal could be achieved by optimizing aeration design utilizing a single stage combined sludge system with the following parameters:

- Aeration MLSS: 1700 to 2300 mg/L,
- Aeration dissolved oxygen: 3.0 mg/L minimum,
- Aeration detention time: 8 hours minimum,
- SRT: 10 to 14 days,
- F/M Ratio: 0.15 to 0.25 kg BOD/kg MLVSS.

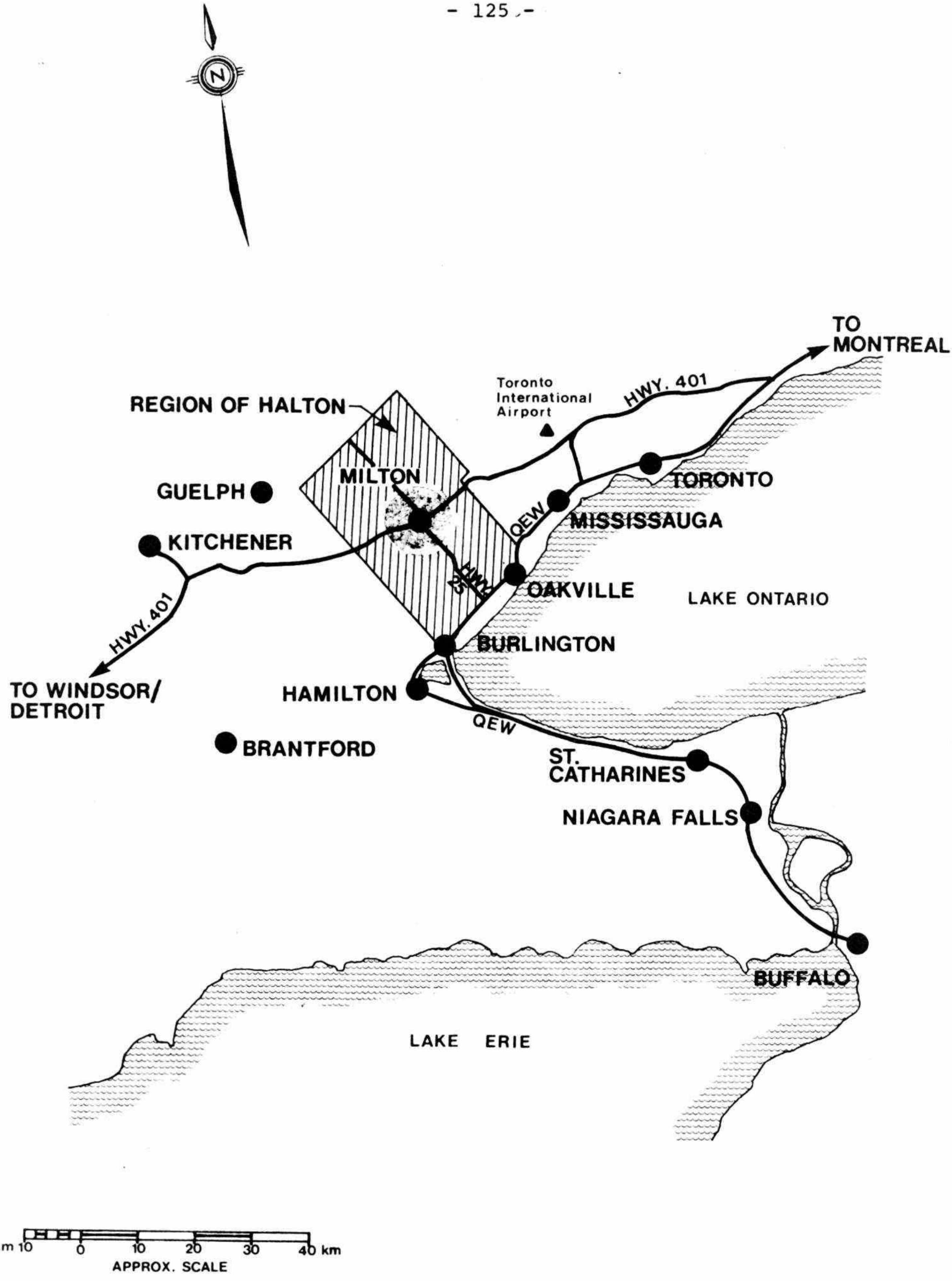

TO MONTREAL
HWY. 401
Toronto International Airport
REGION OF HALTON
MILTON
GUELPH
KITCHENER
TORONTO
MISSISSAUGA
QEW
OAKVILLE
HWY 25
LAKE ONTARIO
HWY. 401
TO WINDSOR/ DETROIT
BURLINGTON
HAMILTON
QEW
BRANTFORD
ST. CATHARINES
NIAGARA FALLS
BUFFALO
LAKE ERIE
km 10 0 10 20 30 40 km
APPROX. SCALE

FIGURE 1

In 1949 the original water pollution control plant for the Town of Milton (Plant "A") first began operation with a treatment capacity of 772 cubic meters per day (m^3/d). Subsequent extensions in 1958 (Plant "B"), 1966 (Plant "B") and 1973 (Plant "C") increased plant capacity to 7176 m^3/d. Further growth in the town resulted in a further extension in 1978 (Plant "D") to increase treatment capacity to 12910 m^3/d.

The combined 7176 m^3/d facility, prior to the most recent expansion, consisted of a conventional activated sludge plant with screening, grit removal, primary settling, diffused aeration, secondary settling, chlorination, phosphorus removal and anaerobic sludge digestion.

With the application of the nitrification criteria, the existing plant capacity was derated from 7176 m^3/day to 5045 m^3/day and a new 7865 m^3/d extension (Plant "D") added so as to achieve the overall total capacity of 12910 m^3/day. The equivalent population served is 28400 people.

The raw sewage design parameters were as follows:

Design average daily flow	12910 m^3/d	28mgd
Design peak daily flow	32277 m^3/d	
BOD_5	200 mg/L	
Suspended Solids	200 mg/L	
TKN average	36 mg/L	
TKN peak	58 mg/L	
Alkalinity	280 mg/L	
Phosphorus	8 mg/L.	

As noted on Figure 2, the upgraded plant incorporates the activated sludge process for secondary treatment for BOD removal and nitrogen conversion preceded by inlet screening, grit removal and primary settling. In addition, effluent filtration, chlorination, dechlorination and phosphorus removal are provided. Sludge treatment facilities consist of 2-stage anaerobic digestion for primary sludges and a multi-stage aerated sludge system with sludge thickening for all waste activated sludges.

The site orientation of each unit process is as noted on Figure 3.

The primary settling tanks are of conventional rectangular design with surface settling rates less than the maximum value of 29.4 $m^3/m^2/d$.

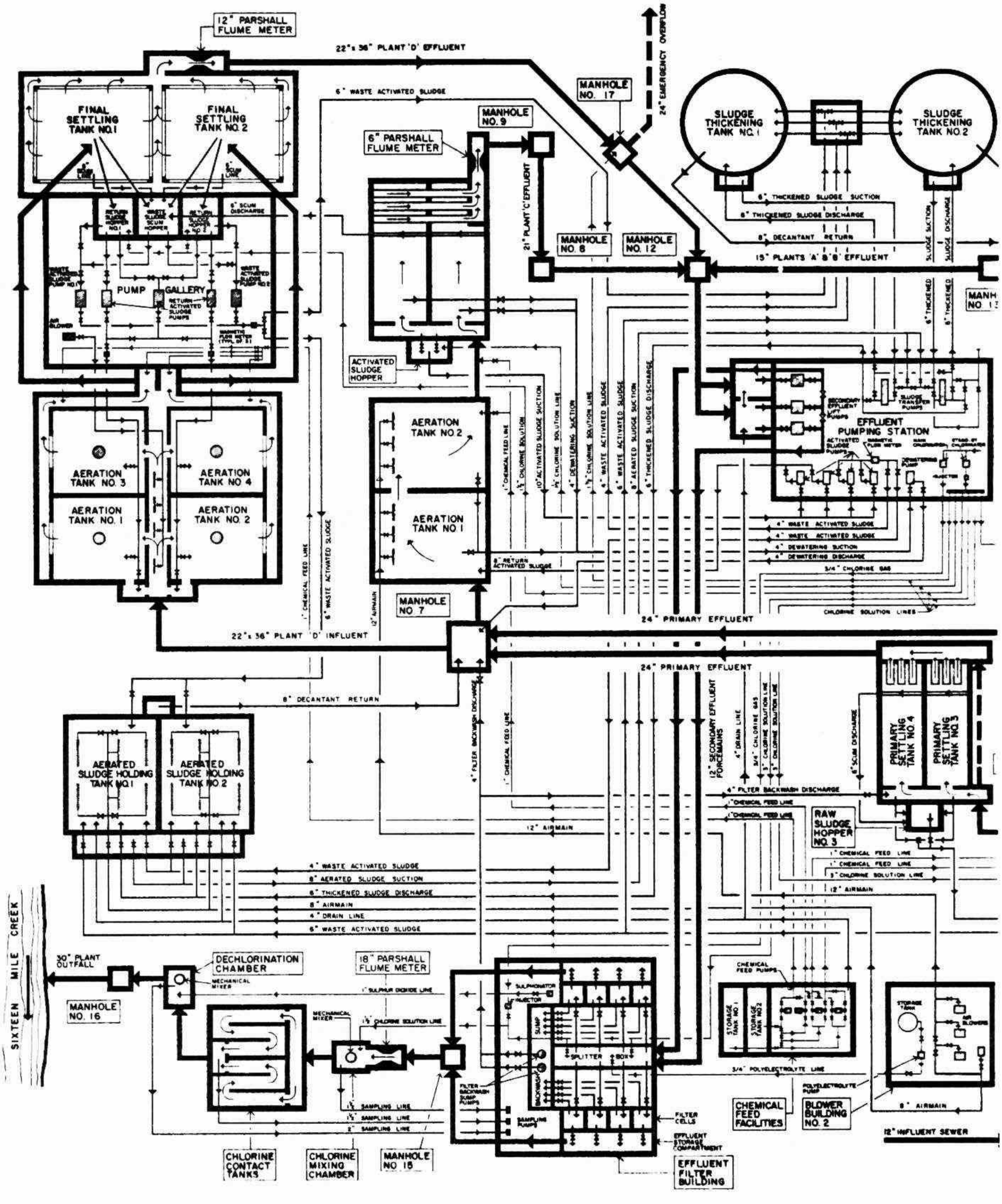
Although existing aeration tanks were fitted with coarse-bubble air diffusion systems, it was concluded that the four new aeration tanks in Plant "D", which account for 61% of total plant capacity, be fitted with mechanical aerators. Draft-tube units, which are characteristically sensitive to water level variations relative to oxygen transfer and power consumption, were considered to offer an optimum degree of flexibility for the nitrification process. Adjustable effluent weirs on each of the four aeration cells provide water level control, subject to power draw with indication on panel-mounted ammeters. As noted on Figures 4 and 5, the new aeration tank configuration permits the system to be operated on the "plug flow" or "complete mix" principle. Return sludge can be introduced in any one of three modes:

- directly into each aeration cell,
- into the aeratin tank inlet trough,
- directly into the lead (head) aeration tank.

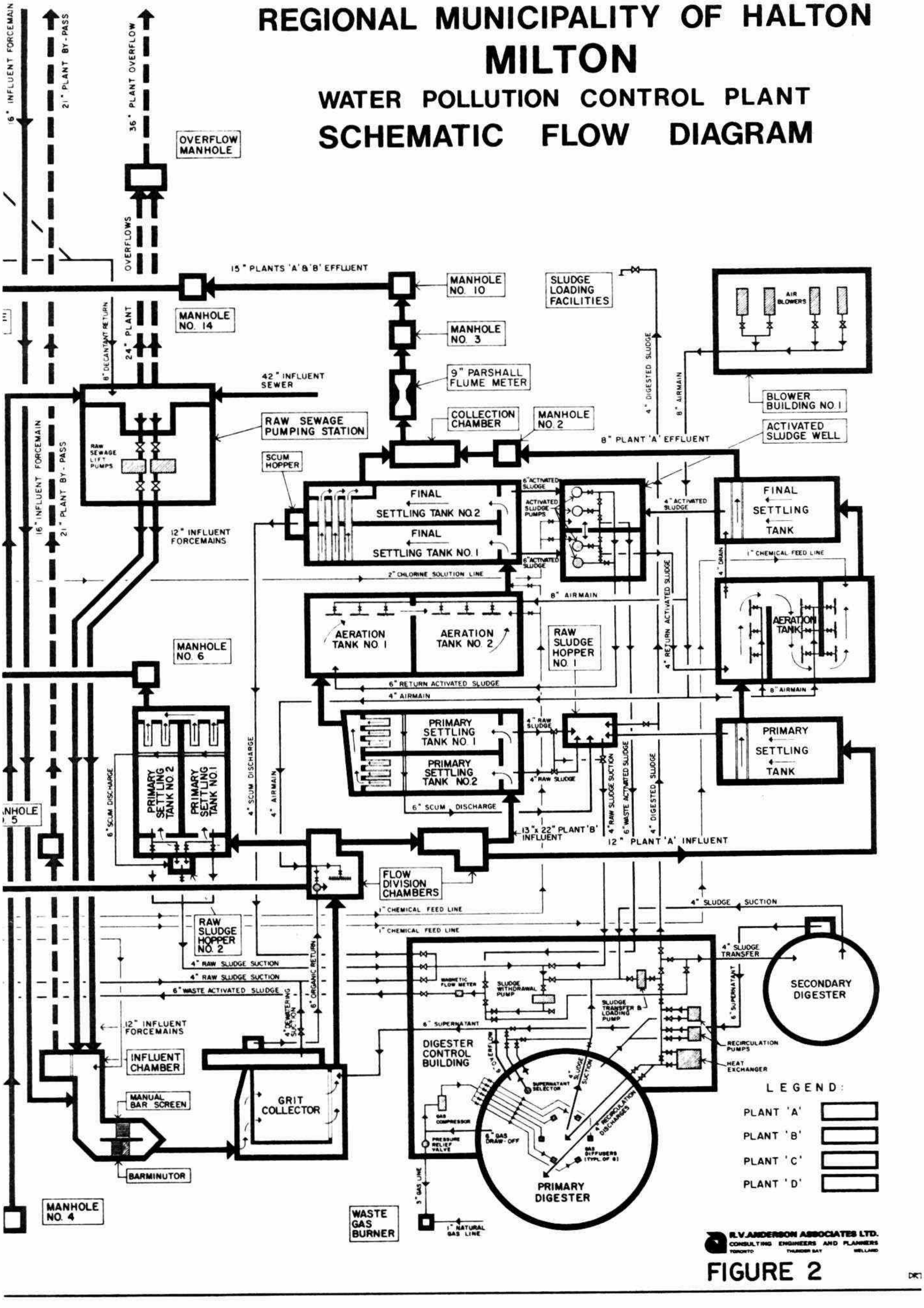
Each of the new aeration cells are 13400 mm square by 4600 mm deep with a volume of 820 m^3 and include a 30 kW bridge mounted mechanical aerator with a maximum output of 55 kg of oxygen per hour. The aerators are designed to satisfy nitrogenous and carbonaceous oxygen demand. The detention time is 10 hours at design flow exclusive of return sludge. With an F/M ratio of 0.20 kg BOD/kg MLVSS, the MLVSS was calculated to be approximately 2300 mg/L. SRT is 10 days.

Based on alkalinity loss of approximately 7 mg/L per mg of ammonia nitrogen oxidized, the buffering capacity of the raw wastewater (alkalinity 280 mg/L$^{\pm}$) was considered adequate to sustain the required pH level for nitrification.

In accordance with the recommendations of the pilot study for a quick return of the settled sludge, the new final clarifiers in Plant "D" were designed with "rapid sludge" removal capability. Due to site


12" PARSHALL FLUME METER
22" x 36" PLANT 'D' EFFLUENT
6" WASTE ACTIVATED SLUDGE
FINAL SETTLING TANK NO. 1
FINAL SETTLING TANK NO. 2
6" SCUM LINE
6" SCUM LINE
RETURN SLUDGE HOPPER NO. 1
WASTE SLUDGE SCUM HOPPER
RETURN SLUDGE HOPPER NO. 2
6" SCUM DISCHARGE
WASTE ACTIVATED SLUDGE PUMP NO. 1
WASTE ACTIVATED SLUDGE PUMP NO. 2
PUMP GALLERY
RETURN ACTIVATED SLUDGE PUMPS
AIR BLOWER
MAGNETIC FLOW METER (TYPICAL OF 3)
AERATION TANK NO. 3
AERATION TANK NO. 4
AERATION TANK NO. 1
AERATION TANK NO. 2
MANHOLE NO. 9
6" PARSHALL FLUME METER
ACTIVATED SLUDGE HOPPER
AERATION TANK NO. 2
AERATION TANK NO. 1
21" PLANT 'C' EFFLUENT
MANHOLE NO. 17
24" EMERGENCY OVERFLOW
MANHOLE NO. 8
MANHOLE NO. 12
SLUDGE THICKENING TANK NO. 1
SLUDGE THICKENING TANK NO. 2
6" THICKENED SLUDGE SUCTION
6" THICKENED SLUDGE DISCHARGE
8" DECANTANT RETURN
15" PLANTS 'A' & 'B' EFFLUENT
SLUDGE SUCTION
SLUDGE DISCHARGE
6" THICKENED
6" THICKENED
MANHOLE NO. 13
1" CHEMICAL FEED LINE
1½" CHLORINE SOLUTION
10" ACTIVATED SLUDGE SUCTION
1½" CHLORINE SOLUTION LINE
4" DEWATERING SUCTION
1½" CHLORINE SOLUTION LINE
4" WASTE ACTIVATED SLUDGE
6" WASTE ACTIVATED SLUDGE
8" AERATED SLUDGE SUCTION
6" THICKENED SLUDGE DISCHARGE
SECONDARY EFFLUENT LIFT PUMPS
SLUDGE TRANSFER PUMPS
EFFLUENT PUMPING STATION
ACTIVATED SLUDGE PUMPS
MAGNETIC FLOW METER
MAIN CHLORINATOR
STAND-BY CHLORINATOR
DEWATERING PUMP
INJECTOR
4" WASTE ACTIVATED SLUDGE
4" WASTE ACTIVATED SLUDGE
4" DEWATERING SUCTION
4" DEWATERING DISCHARGE
3/4" CHLORINE GAS
CHLORINE SOLUTION LINES
8" RETURN ACTIVATED SLUDGE
1" CHEMICAL FEED LINE
6" WASTE ACTIVATED SLUDGE
12" AIRMAIN
MANHOLE NO. 7
22" x 36" PLANT 'D' INFLUENT
24" PRIMARY EFFLUENT
24" PRIMARY EFFLUENT
8" DECANTANT RETURN
4" FILTER BACKWASH DISCHARGE
1" CHEMICAL FEED LINE
12" SECONDARY EFFLUENT FORCEMAINS
4" DRAIN LINE
3/4" CHLORINE GAS
3" CHLORINE SOLUTION LINE
3" CHLORINE SOLUTION LINE
6" SCUM DISCHARGE
PRIMARY SETTLING TANK NO. 4
PRIMARY SETTLING TANK NO. 3
AERATED SLUDGE HOLDING TANK NO. 1
AERATED SLUDGE HOLDING TANK NO. 2
4" FILTER BACKWASH DISCHARGE
1" CHEMICAL FEED LINE
1" CHEMICAL FEED LINE
12" AIRMAIN
RAW SLUDGE HOPPER NO. 3
1" CHEMICAL FEED LINE
1" CHEMICAL FEED LINE
3" CHLORINE SOLUTION LINE
12" AIRMAIN
4" WASTE ACTIVATED SLUDGE
8" AERATED SLUDGE SUCTION
6" THICKENED SLUDGE DISCHARGE
8" AIRMAIN
4" DRAIN LINE
6" WASTE ACTIVATED SLUDGE
SIXTEEN MILE CREEK
30" PLANT OUTFALL
MANHOLE NO. 16
DECHLORINATION CHAMBER
MECHANICAL MIXER
18" PARSHALL FLUME METER
1" SULPHUR DIOXIDE LINE
MECHANICAL MIXER
1½" CHLORINE SOLUTION LINE
1½" SAMPLING LINE
1½" SAMPLING LINE
2" SAMPLING LINE
SULPHONATOR
INJECTOR
SUMP
SPLITTER BOX
BACKWASH
FILTER BACKWASH SUMP PUMPS
SAMPLING PUMPS
FILTER CELLS
EFFLUENT STORAGE COMPARTMENT
CHEMICAL FEED PUMPS
STORAGE TANK NO. 1
STORAGE TANK NO. 2
STORAGE TANK
AIR BLOWERS
3/4" POLYELECTROLYTE LINE
POLYELECTROLYTE PUMP
8" AIRMAIN
12" INFLUENT SEWER
CHLORINE CONTACT TANKS
CHLORINE MIXING CHAMBER
MANHOLE NO. 15
EFFLUENT FILTER BUILDING
CHEMICAL FEED FACILITIES
BLOWER BUILDING NO. 2

REGIONAL MUNICIPALITY OF HALTON
MILTON
WATER POLLUTION CONTROL PLANT
SCHEMATIC FLOW DIAGRAM



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TORONTO THUNDER BAY WELLAND

FIGURE 2

REGIONAL MUNICIPALITY OF HALTON
MILTON
WATER POLLUTION CONTROL PLANT
GENERAL LAYOUT AND FLOW DIAGRAM

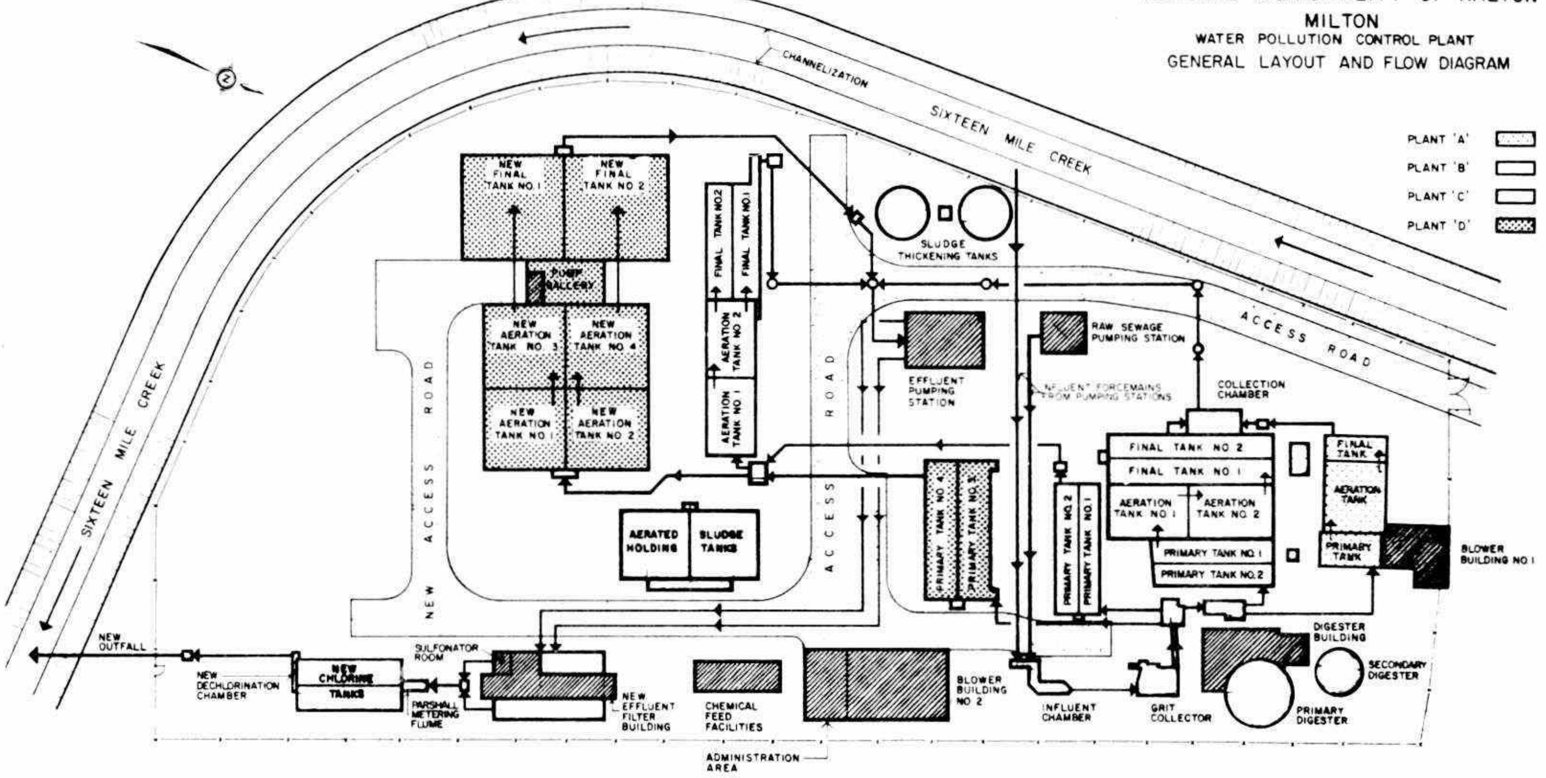

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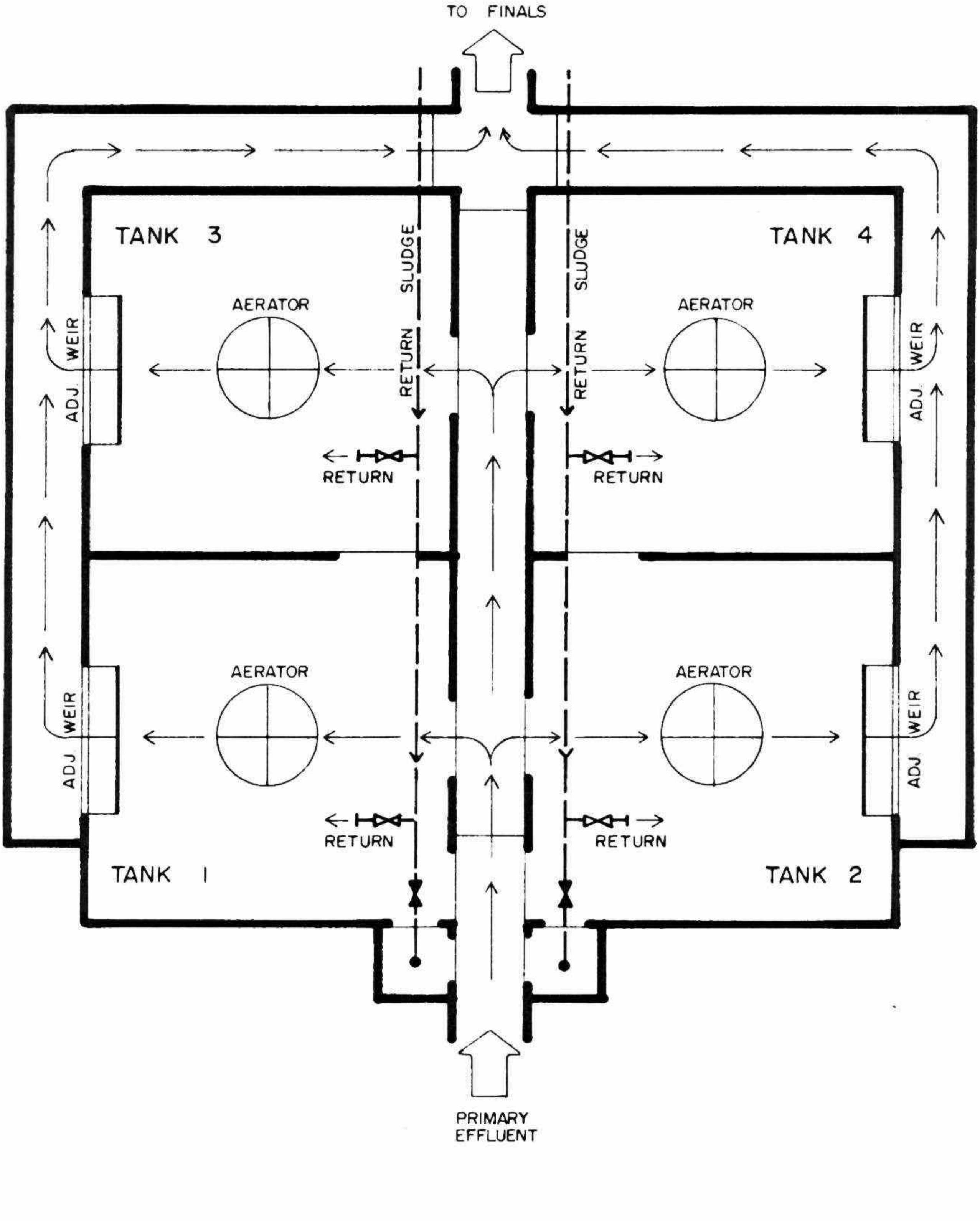
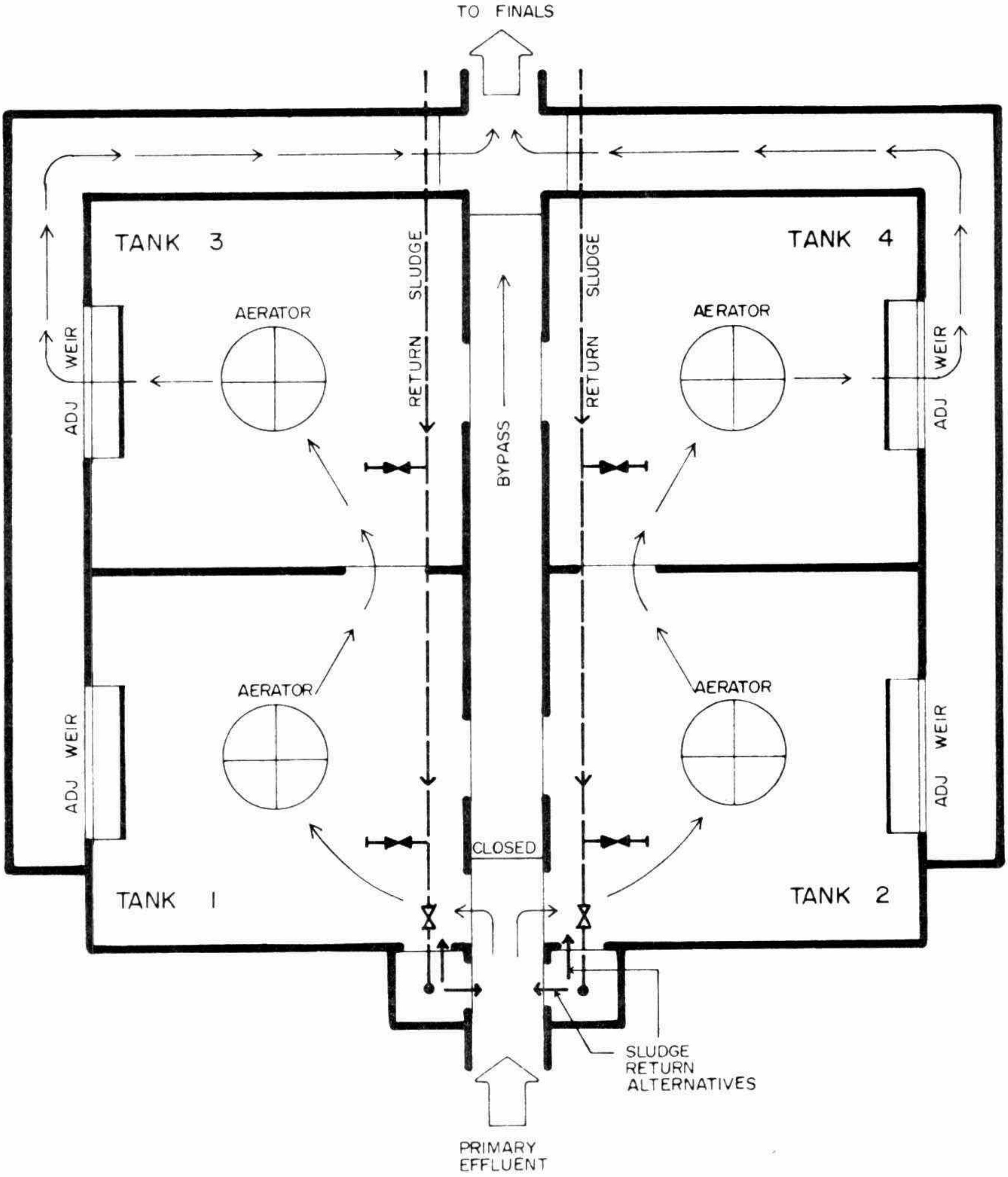
FIGURE 3



PLANT 'D'
COMPLETE MIX
FIGURE 4



PLANT 'D'
PLUG FLOW SERIES
FIGURE 5

restrictions and in order to maximize the use of common wall construction, square type clarifiers with scum removal facilities were selected. There are two new clarifiers and each has a total of six suction lift pipes to carry the sludge to a central collection well for rapid return to the areation tanks. An activated sludge return rate of 150% can be achieved. Based on an empirical design average SVI of 143 (7000 mg/L return sludge concentration - less than conventional because of rapid sludge removal feature), the calculated return rate is in the order of 49% for the new plant. Each clarifier is 17070 mm square x 3810 mm side water depth. The surface settling rate at peak flow is 33.8 $m^3/m^2/d$.

All return and waste sludge is gauged by magnetic flow meters prior to return to the system. A summary of the design criteria used for the clarification and secondary treatment process is as noted on Table 1.

Due to the limited capacity of the existing 2-stage anaerobic digesters, which employ gas mixing and available land restraints, it was decided to employ a separate aerobic stabilization system for all waste activated sludges thereby utilizing existing tanks. Due to the significant dissolved oxygen levels in the waste activated sludge as a result of the nitrification regime, aerobic stabilization was considered a significant advantage. The anerobic digestion system is intended to handle a design load of 1547 kg/day whereas the aerobic system will acommodate a load of 1295 kg/day. There are two aerated sludge holding tanks each 11277 mm x 11277 mm x 4267 mm side water depth.

The aerobic sludge is concentrated in circular sludge thickening tanks prior to truck disposal. The system is so designed that waste activated sludge can be thickened prior to or after aerobic digestion. The aerobic digesters also have the facility to permit supernatant withdrawal within the tank.

The chemical feed system for phosphorus removal includes four positive displacement chemical metering pumps, each automatically controlled by an electronic signal from the respective effluent flow meters from each of the plants. Chemical storage consists of two tanks, each with a capacity of 55000 liters and are insulated and electrically heated.

System flexibility will permit the use of either ferric chloride or liquid alum. Alum is presently used.

With an effluent objective of 4.2 mg/L BOD and 0.43 mg/L phosphorus, the nitrified secondary effluent is introduced to a tertiary filter system. Effluent from the four secondary plant streams gravitates to a central pumping station which includes three variable speed, automatically-controlled pumps each rated at 16138 m^3/day. At the filter building, the pumped effluent discharges into a splitter box and is uniformally proportioned to eight gravity-type filter cells with anthracite and sand filter media. The sand layer is 300 mm deep with a gradation of 0.45 to 0.55 mm and a specific gravity greater than 2.5. Above the sand media, a layer of anthracite is placed to a depth of 450 mm with a gradation of 1.0 to 1.2 mm and a specific gravity of at least 1.5. The filter has a self contained automatic backwash system with air scour. With a total filter area of 114.5 m^2 the maximum filtration rate at peak flow is 282 m^3/m^2/day. A fully automated control system provides automatic backwash control based on loss-of-head or preset time intervals. Backwash water is stored in a sump and slowly pumped back to the aeration tanks. As a filter aid, provision has been made for application of a polyelectrolyte solution. Chlorine can also be added prior to filtration. In the basement pipe gallery, each filter cell, of reinforced concrete construction, is fitted with a 25 mm thick observation window which provides operating personnel and visitors with an interesting view of the filtration process. A dehumidification system serves to minimize condensation problems in the filter building. The filtration building also houses the sulphonator and chlorination/dechlorination control system.

Immediately downstream from the filter system, the effluent passes through a 460 mm parshall flume to the chlorine contact tank. The tank provides a detention period of 15 minutes at peak flow and incorporates a serpentine baffle system to ensure optimum velocities and reduce short-circuiting. To further optimize chlorine mixing, a propeller-type mechanical mixer is located in a small mixing chamber at the discharge end of the parshall flume. The chlorine tank is also fitted with isolation gates to permit one-half of the tank to be dewatered and cleaned.

In order to provide a minimum chlorine residual of 0.5 mg/L for pathogenic disinfection but at the same time meet a maximum residual objective of 0.08 mg/L to the receiving stream, sulfur dioxide solution in introduced at the outlet of the chlorine tank. The dechlorination chamber provides approximately 2.5 minutes detention at design flow and is equipped with a propeller-type mixer and inlet/outlet baffling to ensure optimum mixing of the sulfur dioxide solution. Subject to field adjustment, a dosage rate of 1.0 kg sulfur dioxide per 1.0 kg chlorine has been allowed for in the design.

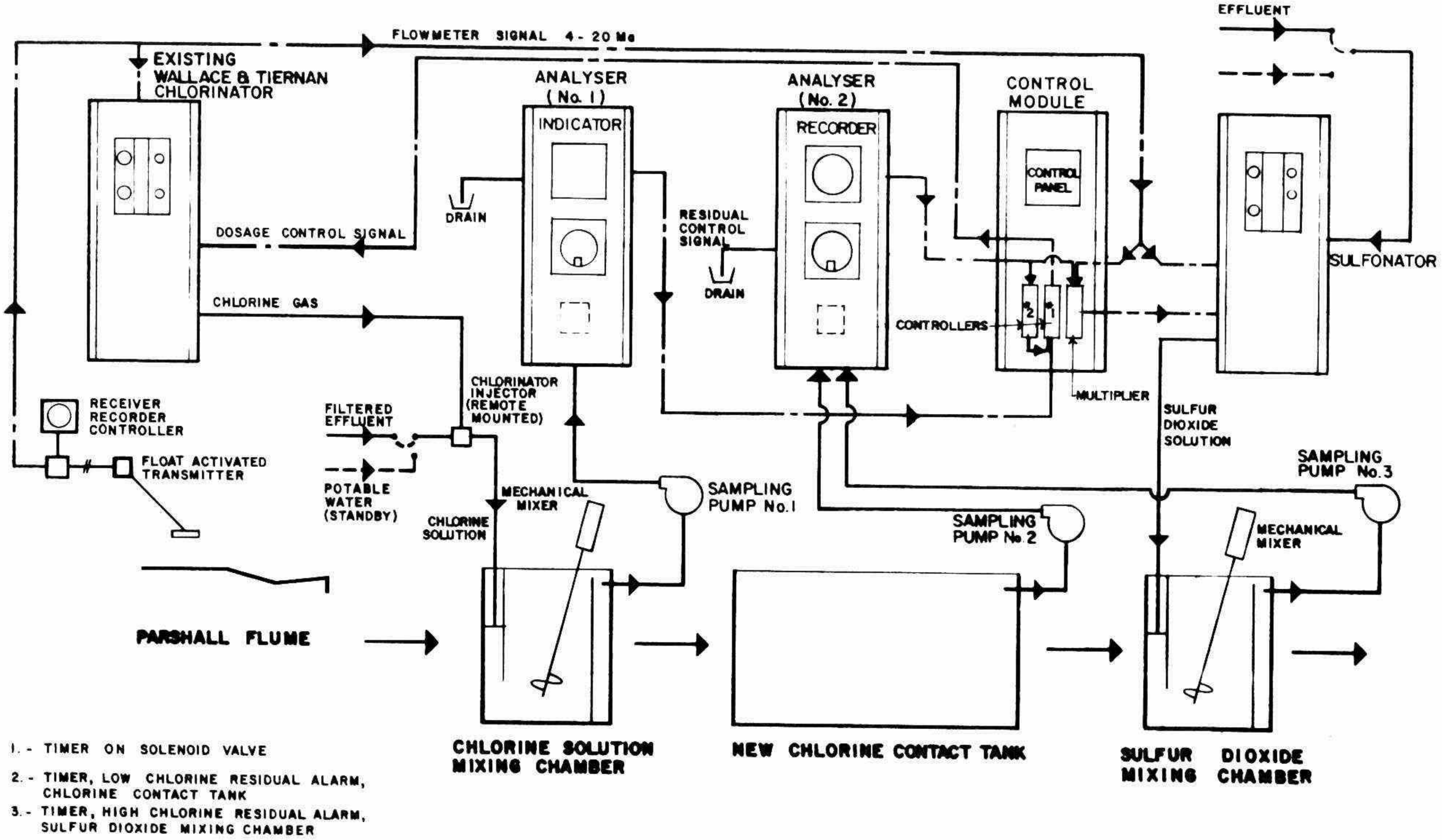
The chlorination/dechlorination sampling flow schematic and control logic is as noted on Figure 6.

Using "compound loop technology", the system includes two residual analyzers, two flow controllers, chart recorders and time control circuitry. Three variable-speed, positive displacement sampling pumps, located in the basement of the filter building, in close proximity to the chlorine tank, provide a continuous supply of sample effluents to the control system. It is important to note that in order to reduce the "lag" time, the sampling pumps should be located as close as possible to the point of sampling and analyzers.

Basically the chlorination/dechlorination system is automatically controlled to allow for variations in flow rate and chlorine demand.

The chlorination system is controlled by a flow signal and a residual signal. The flow signal from the total plant flow recorder controls the chlorine solution feed in the chlorine mixing chamber in proportion to the flow. A continuous sample from the mixing chamber is pumped to Analyser No. 1 by one of the sample pumps. Analyser No. 1 determines the chlorine residual of the sample and compares it to a desired residual "set point" in Controller No. 1. This set point is varied by the second controller based on the chlorine demand as determined by the Analyser No. 2. If the chlorine residual and the set point are not equal a correcting signal is transmitted to the chlorinator to increase or decrease the chlorine feed.

REGIONAL MUNICIPALITY OF HALTON
MILTON WATER POLLUTION CONTROL PLANT
CHLORINATION / DECHLORINATION SYSTEM
GENERAL SCHEMATIC



1. - TIMER ON SOLENOID VALVE
2. - TIMER, LOW CHLORINE RESIDUAL ALARM, CHLORINE CONTACT TANK
3. - TIMER, HIGH CHLORINE RESIDUAL ALARM, SULFUR DIOXIDE MIXING CHAMBER

FIGURE 6

A second sample is pumped from the discharge end of the chlorine tank to Analyser No. 2 to determine the residual of the sample and compares it in Controller No. 2 to the desired residual of 0.5 mg/L for proper disinfection. This residual is called the "manual setpoint" which is adjusted by the operator if the actual measured residual is less than or more than the set point. Another correcting signal from Controller No. 2 increases or decreases the variable setpoint of Controller No. 1 which is the desired residual at the chlorine mixing chamber.

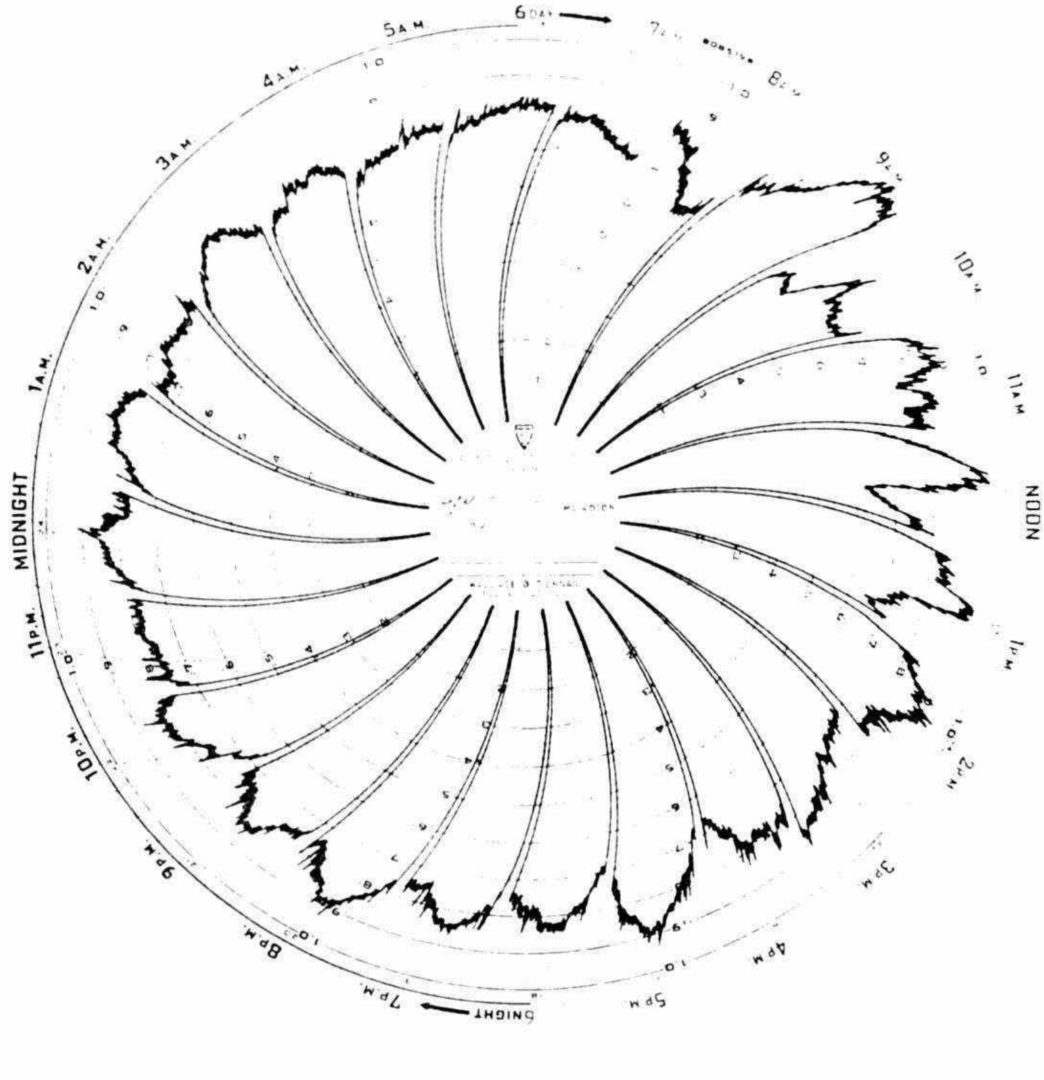
The chlorine residual signal from Analyser No. 2 is fed into a multiplier and combine it with a flow signal from the parshall flume. The combined signal is used to regulate the sulfonator feed rate.

Time delays are built in the controllers to compensate for delays between adding the chlorine and sensing the residual change caused by the change in dosage. Analyser No. 2 is used to check both the residual from the chlorine contact tank and the dechlorinated effluent. A timer would allow the measurement of the chlorinated effluent residual for 55 minutes and the dechlorinated effluent residual for 5 minutes. This proportion in time need to be changed in order to allow enough time for recording the dechlorinated effluent residual.

A chart recorder gives a continuous read out of the actual system performance. Figure 7 is a typical chart read out. Alarm circuitry was designed into the system to respond to residual levels in excess of 0.08 mg/L or if the disinfection levels in the chlorine tank drop below 0.50 mg/L.

For some time now, the operation of the chlorination/dechlorination control system has experienced problems associated with the controller adjustments and automatic signals. The result has been, at times, a somewhat erratic control on the 0.5 mg/L chlorine residual prior to dechlorination. The system requires constant calibration. Approximately 1 man hour per day is required to maintain the chlorination/dechlorination instrumentation system.

REGIONAL MUNICIPALITY OF HALTON
MILTON POLLUTION CONTROL PLANT
DECHLORINATION WITH SULFUR DIOXIDE



NORMAL CHLORINE RESIDUAL PRIOR TO DECHLORINATION
WITH HOURLY REFERENCE TO FINAL DECHLORINATED EFFLUENT

FIGURE 7

Of note with the chlorination/dechlorination system is the cost of chemicals for the residual chlorine analysis as compared to the actual chlorine and sulphur dioxide used. The 1980 costs can be summarized as follows:

- Chlorine	$5,460
- Sulphur Dioxide	$3,635
- Buffer for Residual Chlorine Analysis	$2,400

Design and field experience would suggest that future chlorination/dechlorination control system designs should address, in particular, the following:

1. Ensure compatability of the various components and the effects on any measurement delays in their loops.

2. Carefully consider the state of the art technology with respect to buffer usage for chlorine residual analysis.

3. The effect on the control system due to a reduction in flow to the chlorination/dechlorination tanks during filter backwash if tertiary filtration is employed upstream of the chlorine tank.

4. The specific daily operator attention required.

5. The affects on the control circutry of frequent manual operator adjustment of the chlorine "set points".

Resulting largely from the critical nature of the receiving stream and the need to provide maximum treatment capacility, at all times, the plant is equipped with 2 diesel-driven power generators, with a total capacity of 625 kva.

The Region has a regular sampling and monitoring program.

During the months of July and August 1980, an extensive testing program was undertaken by Pollutech. In order to achieve maximum loading conditions, all the flow was directed, after primary treatment, to the aeration section of the newly completed Plant "D" with a rated capacity of 7865 m^3/d. The average daily flow during the test period was approximately 6600 m^3/d (i.e. 84% of design capacity of Plant "D"). Raw sewage characteristics during the test period are noted on Table 2A. The results of this testing program are as noted on Tables 2, 3, and 4. Samples were flow proportioned 24 hour composites.

Table 2 indicates the alkalinity, TKN and NH_3 removal across the primary settling tanks* with average values for the test period as follows:

- primary tank alkalinity loss of 16 mg/L from 334 to 318 mg/L,
- primary tank TKN loss of 5.2 mg/L from 24.3 to 19.1 mg/L,
- primary tank NH_3-N loss of 2.7 mg/L from 16.6 to 13.9 mg/L.

The TKN and NH_3 removals across the entire plant are as noted in Table 3 with average values as follows:

TKN (mg/L)		NH_3-N (mg/L)		Percent Removal	
Raw	Final	Raw	Final	TKN	NH_3-N
24.3	0.99	16.6	0.63	96%	96%

Table 4 indicates the final plant effluent during the study period with average values as follows:

BOD_5	1.6 mg/L
SS	3 mg/L
pH	7.5
Alkalinity	186 mg/L
TKN	0.99 mg/L
NH_3-N	0.63 mg/L
NO_2-N + NO_3-N	8.2 mg/L

* combined primary settling design capacity of Plants "C" and "D" of 9683 m^3/d utilized during July/August 1980 test period for the period flow of 6600 m^3/d.

Table 5 notes some monthly soluble phosphorus values for the raw sewage and effluent for the year 1980.

The winter 1980 nitrification values for the months of February, March, April were extracted from the operating data and are as noted on Table 6. It would appear that nitrification was occurring under the conditions prevalent at that time. It should be noted that all 4 plants were in service with an average daily flow of approximately 6310 m^3/d vs. the design flow of 12910 m^3/d (i.e. detention times would be proportionately longer in the aeration tanks).

The plant has a modern, well equipped laboratory enabling the plant staff to undertake the majority of analysis necessary for monitoring and operational control. At the present time, the Regional Municipality of Halton maintains a normal operating staff of four men.

Construction took place over a period of approximately 15 months at a total cost of $2,831,000. During this period, the contractor was required to closely co-ordinate his construction operations so as to allow uninterrupted operation of existing plant facilities. To further complicate construction of this project, Halton Region Conservation Authority simultaneously completed construction of concrete-lined channelization for Sixteen Mile Creek which immediately abutted the plant site. However, excellent co-operation between the respective contractors and the owner served to minimize problems in this area.

(1) Mr. D. R. Morrier, P. Eng., Manager, Operations and Maintenance, The Regional Municipality of Halton, Oakville, Ontario.

(2) Mr. P. J. Laughton, P. Eng., Director of Environmental Engineering, Anderson Associates Limited, Consulting Engineers, Willowdale, Ontario.

(3) Mr. S. N. Gergis, P. Eng., Senior Project Engineer, Anderson Associates Limited, Consulting Engineers, Willowdale, Ontario.

TABLE 1

REGIONAL MUNICIPALITY OF HALTON

MILTON WATER POLLUTION CONTROL PLANT

SUMMARY OF DESIGN PARAMETERS

DESIGN CRITERIA	PLANT 'A'	PLANT 'B'	PLANT 'C'	NEW PLANT 'D'	LIMITS
DESIGN FLOW - M^3/D	500	2727	1818*	7865**	-
PRIMARY S.S. RATE - M^3/D/M^2	27.46	20.38	25.67	29.0	29.34 MAX.
PRIMARY DETENTION - HOURS	2.28	2.51	2.51	2.52	2.5 MIN.
AERATION LOAD - G BOD/M^3	184	340	340	320	-
AERATION MLSS - MG/L	1600	2285	2289	2305	1700 - 2300
AERATION DETENTION - HOURS	17.35	9.44	9.42	10.02	8.0 MIN.
SLUDGE AGE (EPA DEFINITION)	14.8	11.2	11.2	10	10 - 14
F/M RATIO - G BOD/G MLVSS	0.16	0.21	0.21	0.20	0.15 - 0.25
RETURN SLUDGE - % @ SVI OF 143	29.6	48.5	48.6	49.1	-
FINAL S.S. RATE - M^3/D/M^2 PEAK FLOW	33.52	34.74	32.44	33.81	34.25 MAX.
FINAL DETENTION - HOURS	5.9	5.8	6.2	6.8	-
FINAL SOLIDS LOAD - KG/D/M^2	21.40	31.70	29.3	31.10	73.35 MAX.

NOTES

- ABOVE CRITERIA BASED ON AVERAGE DWF WITH RECOGNITION OF 2.5 PEAKING FACTOR, UNLESS OTHERWISE NOTED.
- *1818 M^3/D THROUGH AERATION AND FINAL STAGES, 3863 M^3/D THROUGH EXISTING PRIMARY TANKS.
- **7865 M^3/D THROUGH AERATION AND FINAL STAGES, 5815 M^3/D THROUGH NEW PRIMARY TANKS.

TABLE 2A

REGIONAL MUNICIPALITY OF HALTON

MILTON WATER POLLUTION CONTROL PLANT

RAW SEWAGE CHARACTERISTICS* FOR JULY/AUGUST 1980 TEST PERIOD

PARAMETER (MG/L)

DATE	BOD_5	SS	pH	ALKALINITY	TKN	NH_3-N	NO_2-N + NO_3-N
JULY 21, 1980	98	105	8.2	358	31.9	22.8	0.5
JULY 23, 1980	153	206	7.7	324	23.5	15.1	0.5
JULY 25, 1980	74	96	7.4	328	21.7	19.3	0.4
JULY 28, 1980	107	107	7.2	348	25.9	16.5	0.3
JULY 30, 1980	79	93	7.2	314	20.4	11.3	0.6
AUGUST 1, 1980	126	163	7.5	334	22.1	14.8	1.2
AVERAGE	106	128	7.5	334	24.3	16.6	0.6

* FLOW WEIGHTED COMPOSITES

TABLE 2

REGIONAL MUNICIPALITY OF HALTON

MILTON WATER POLLUTION CONTROL PLANT

ALKALINITY, TKN, NH$_3$-N REMOVAL ACROSS PRIMARY CLARIFICATION

PARAMETER (MG/L)

DATE	ALKALINITY$_{(I)}$	ALKALINITY$_{(E)}$	TKN$_{(I)}$	TKN$_{(E)}$	NH$_3$-N$_{(I)}$	NH$_3$-N$_{(E)}$
JULY 21, 1980	358	336	31.9	21.0	22.8	17.5
JULY 23, 1980	324	310	23.5	18.8	15.1	15.2
JULY 25, 1980	328	300	21.7	17.5	19.3	13.5
JULY 28, 1980	348	332	25.9	25.9	16.5	16.0
JULY 30, 1980	314	300	20.4	16.6	11.3	8.8
AUGUST 1, 1980	334	330	22.1	14.9	14.8	12.8
AVERAGE	334	318	24.3	19.1	16.6	13.9

PERCENT REMOVAL ACROSS PRIMARY 4.8% 21.3% 16.3%

(I)	INFLUENT	AVERAGE DAILY FLOW JULY 1980 6836 M^3.D
(E)	EFFLUENT	DESIGN AVERAGE FLOW PRIMARY CLARIFIERS PLANTS C,D IN SERVICE DURING TEST PERIOD 9683 M^3.D

NOTE: ALL SAMPLES FLOW WEIGHTED COMPOSITES

TABLE 3

REGIONAL MUNICIPALITY OF HALTON

MILTON WATER POLLUTION CONTROL PLANT

TKN AND NH_3 REMOVALS ACROSS PLANT

PARAMETER (MG/L)

	TKN		NH_3-N		NO_2-N + NO_3-N		PERCENT REMOVAL	
DATE	RAW	FINAL*	RAW	FINAL*	RAW	FINAL*	TKN	NH_3-N
JULY 21, 1980	31.9	0.98	22.8	0.88	0.5	10.2	97	96
JULY 23, 1980	23.5	1.49	15.1	1.30	0.5	9.0	94	91
JULY 25, 1980	21.7	0.96	19.3	0.73	0.4	8.8	96	96
JULY 28, 1980	25.9	0.80	16.5	0.23	0.3	8.0	97	97
JULY 30, 1980	20.4	0.91	11.3	0.43	0.6	5.6	96	96
AUGUST 1, 1980	22.1	0.80	14.8	0.18	1.2	7.4	96	99
AVERAGE	24.3	0.99	16.6	0.63	0.6	8.2	96%	96%

* TO CREEK

NOTE: ALL SAMPLES FLOW WEIGHTED COMPOSITES

TABLE 4

REGIONAL MUNICIPALITY OF HALTON

MILTON WATER POLLUTION CONTROL PLANT

FINAL PLANT EFFLUENT TO CREEK

PARAMETER (MG/L)

DATE	BOD$_5$	SS	PH	ALKALINITY	TKN	NH$_3$-N	NO$_2$-N + NO$_3$-N	FLOW* M^3.D
JULY 21, 1980	2.1	4	8.2	186	0.98	0.88	10.2	6596
JULY 23, 1980	1.1	5	7.6	188	1.49	1.30	9.0	5618
JULY 25, 1980	1.9	3	7.3	194	0.96	0.73	8.8	6410
JULY 28, 1980	0.8	3	7.1	184	0.80	0.23	8.0	6207
JULY 30, 1980	1.8	2	7.4	172	0.91	0.43	5.6	8317
AUGUST 1, 1980	1.8	1	7.1	194	0.80	0.18	7.4	6807
AVERAGE	1.6	3	7.5	186	0.99	0.63	8.2	6659

* PLANT "D" DESIGN 7865 M^3.D

NOTE: ALL SAMPLES FLOW WEIGHTED COMPOSITES

TABLE 5

REGIONAL MUNICIPALITY OF HALTON

MILTON WATER POLLUTION CONTROL PLANT

1980 SOLUBLE PHOSPHORUS REMOVAL

DATE	SOLUBLE PHOSPHORUS (INFLUENT) MG/L	SOLUBLE PHOSPHORUS (EFFLUENT)* MG/L
JANUARY 22	3.4	0.26
FEBRUARY 25	4.4	0.54
MARCH 24	3.6	0.32
APRIL 28	2.8	0.70
MAY 12	3.9	0.41
JUNE 24	4.0	0.44
JULY 28	2.1	0.31
AUGUST 13	4.2	0.42
SEPTEMBER 10	4.1	0.44
OCTOBER 16	3.9	0.24
NOVEMBER 24	4.8	0.28
DECEMBER 8	2.6	0.09

* VARIATION 0.04 MG/L TO 1.0 ± MG/L

TABLE 6

REGIONAL MUNICIPALITY OF HALTON

MILTON WATER POLLUTION CONTROL PLANT

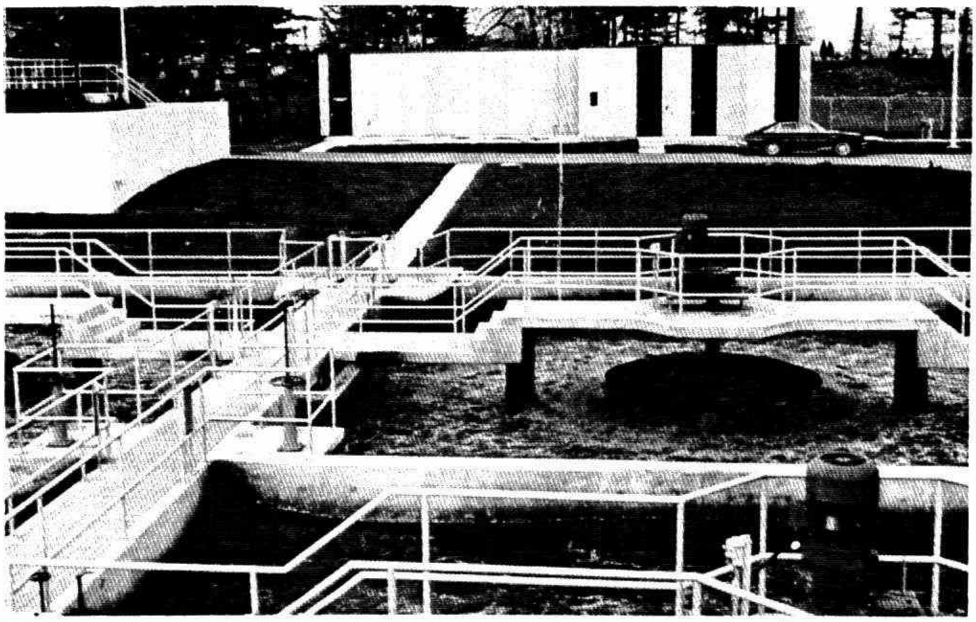
1980 WINTER NITRIFICATION RESULTS

PARAMETER (MG/L)

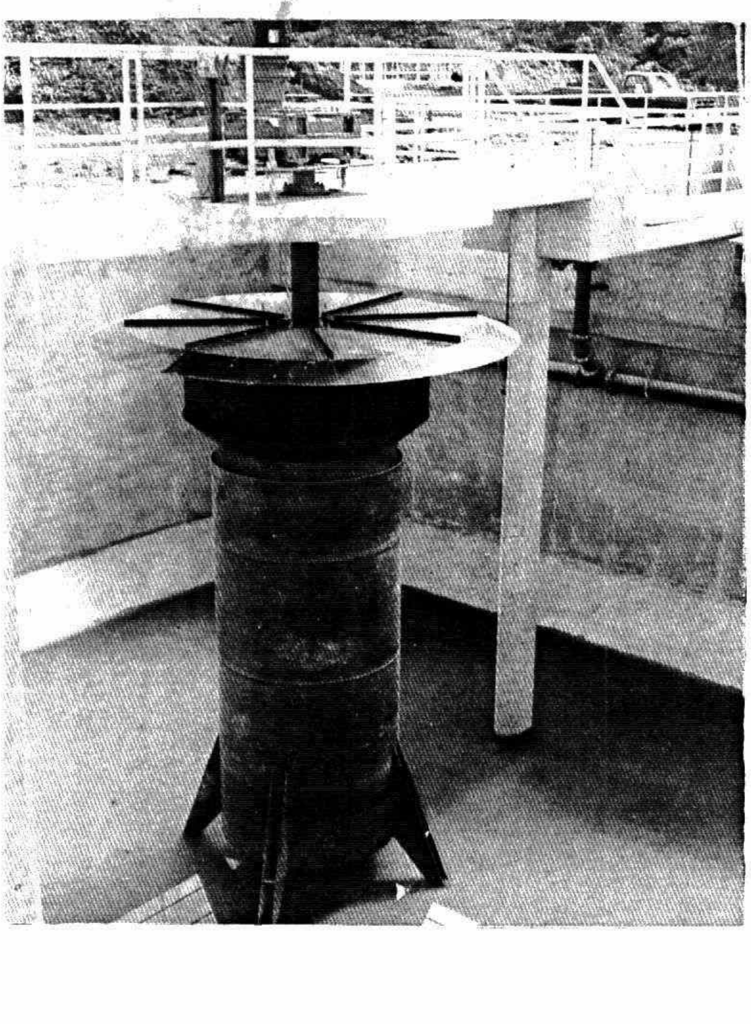
DATE	$BOD_{(I)}$	$BOD_{(E)}$	$TKN_{(I)}$	$TKN_{(E)}$	$NH_3\text{-}N_{(I)}$	$NH_3\text{-}N_{(E)}$
February 12, 1980	264	5	53	2.8	28	<0.1
February 25, 1980	140	4	39	1.2	23	0.2
March 23, 1980	88	0.6	29	2.2	18	1.3
April 28, 1980	192	3	42	6.0	14	0.3

1980 AVERAGE DAILY FLOW = 6310 $M^3.D$
PLANT DESIGN FLOW = 12910 $M^3.D$

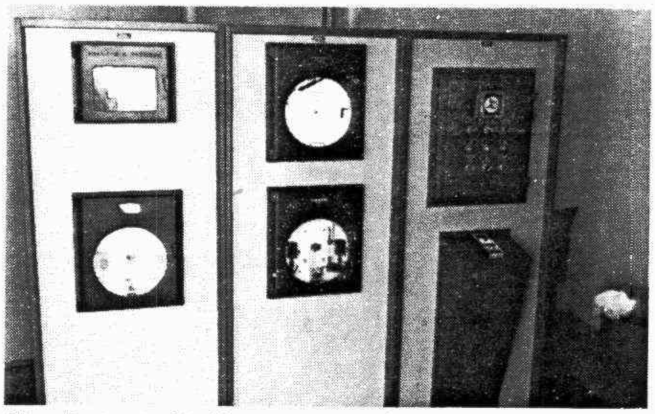
(I) Influent
(E) Effluent



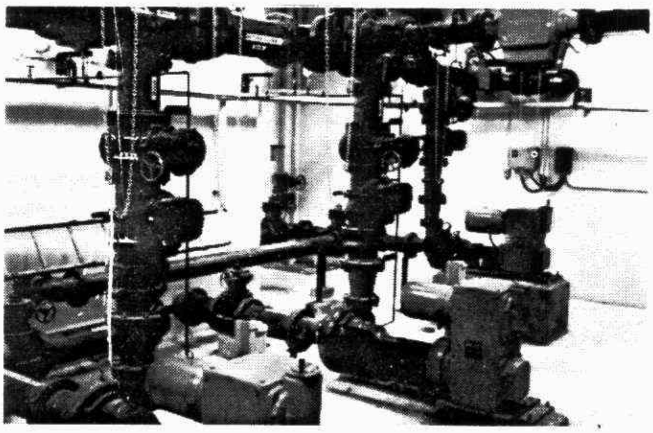
mechanical aerators



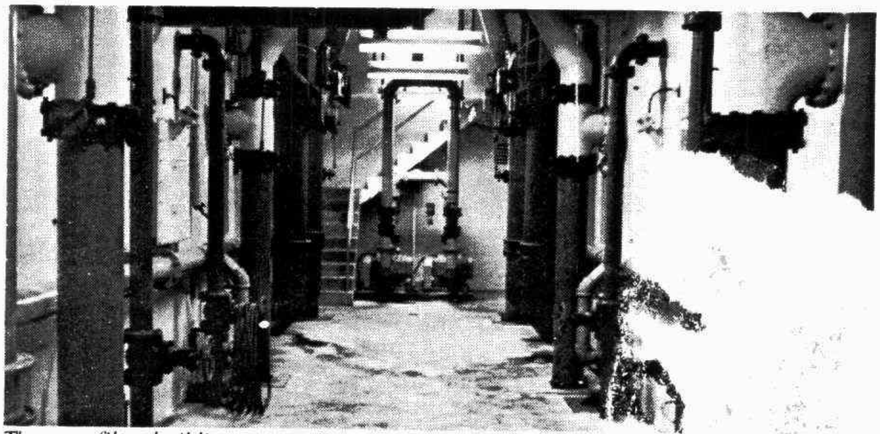
Draft-tube units, which are characteristically sensitive to water level variations relative to oxygen transfer and power consumption, were considered to offer an optimum degree of flexibility for the nitrification process. Adjustable effluent weirs on each of the four aeration cells provide water level control, subject to power draw with indication on panel-mounted ammeters. The new aeration tank configuration permits the system to be operated on the "plug flow" or "complete mix" principle.



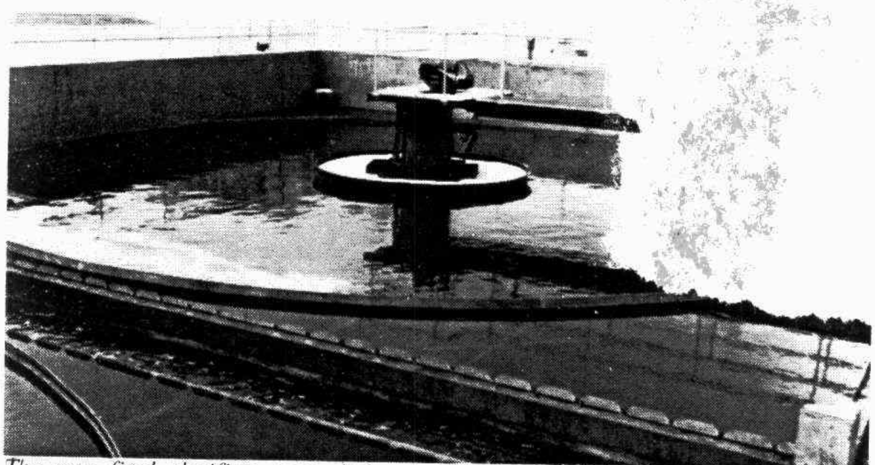
The chlorination/dechlorination control panel, developed because of the complex chlorine parameters on this project, is thought to be unique in municipal Canadian facilities.



New activated return pumps. An activated sludge return rate of 150% can be achieved.



The new filter building pipe gallery. To minimize costs prec walls were used in conjunction with precast roof slabs, f



The new final clarifiers were designed with "rapid sludge" removal capability. Square clarifiers with scum removal facilities were selected.